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NATIONAL DAM SAFETY PROGRAM. STURGEON POOL DAM (INVENTORY NUMBE--ETC(U)
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1 OF 3

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LOWER HUDSON RIVER BASIN

STURGEON POOL DAM

ULSTER COUNTY, NEW YORK
INVENTORY NUMBER NY 75

PHASE 1 INSPECTION REPORT NATIONAL DAM SAFETY PROGRAM

APPROVED FOR PUBLIC RELEASE;
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CONTRACT NO. DACW-51-78-C-0025

Prepared by

L. ROBERT KIMBALL and ASSOCIATES
615 W. Highland Ave. Ebensburg, Pa.

Prepared For

DEPARTMENT OF THE ARMY
NEW YORK DISTRICT, CORPS OF ENGINEERS
NEW YORK, NEW YORK

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U. S. ARMY ENGINEER DISTRICT, NEW YORK
26 FEDERAL PLAZA
NEW YORK, NEW YORK 10007

2 OCT 1..3

NANEN-F

Honorable Hugh L. Carey
Governor of New York
Albany, New York 12224

Dear Governor Carey:

The purpose of this letter is to inform you of a clarification of the guidelines used by this office in assessing dams under the National Program of Inspection of Dams.

Office of the Chief of Engineers has recently provided a clarification that dams with seriously inadequate spillways are to be assessed as unsafe, non-emergency, until more detailed studies prove otherwise or corrective measures are completed.

The following dams in your state have previously been assessed as having seriously inadequate spillways, with capability to pass safely only the percentage of the probable maximum flood as noted in each report. They are now to be assessed as unsafe:

I.D. NO.	NAME OF DAM
N.Y. 59	Lower Warwick Reservoir Dam
N.Y. 4	Salisbury Mills Dam
N.Y. 45	Amawalk Dam
N.Y. 418	Jamesville Dam
N.Y. 685	Colliersville Dam
N.Y. 6	Delta Dam
N.Y. 421	Oneida City Dam
N.Y. 39	Croton Falls Dam
N.Y. 509	Chadwick Dam (Plattenkill)
N.Y. 66	Boyds Corner Dam
N.Y. 397	Cranberry Lake Dam
N.Y. 708	Seneca Falls Dam
N.Y. 332	Lake Sebago Dam
N.Y. 338	Indian Brook Dam
N.Y. 33	Lower(S) Wiccopee Dam (Lower Hudson W.S. for Peekskill)

NANEN-F
Honorable Hugh L. Carey

I.D. NO.	NAME OF DAM
N.Y. 49	Pocantico Dam
N.Y. 445	Attica Dam
N.Y. 658	Cork Center Dam
N.Y. 153	Jackson Creek Dam
N.Y. 172	Lake Algonquin Dam
N.Y. 318	Sixth Lake Dam
N.Y. 13	Butlet Storage Dam
N.Y. 90	Putnam Lake (Bog Brook Dam)
N.Y. 166	Pecks Lake Dam
N.Y. 674	Bradford Dam
N.Y. 75	Sturgeon Pool Dam
N.Y. 414	Skaneateles Dam
N.Y. 155	Indian Lake Dam
N.Y. 472	Newton Falls Dam
N.Y. 362	Buckhorn Lake Dam

The classification of "unsafe" applied to a dam because of a seriously inadequate spillway is not meant to connote the same degree of emergency as would be associated with an "unsafe" classification applied for a structural deficiency. It does mean, however, that based on an initial screening, and preliminary computations, there appears to be a serious deficiency in spillway capacity so that if a severe storm were to occur, overtopping and failure of the dam would take place, significantly increasing the hazard to loss of life downstream from the dam.

Consequently, it is advisable to implement the recommendations previously furnished in the reports for the above-mentioned dams as soon as practicable.

It is requested that owners of these dams be furnished a copy of this letter and that copies be permanently appended to all reports previously furnished to you.

Sincerely yours,

CLARK H. BENN
Colonel, Corps of Engineers
District Engineer

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Ulster County
Wallkill River

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report provides information and analysis on the physical condition of the dam as of the report date. Information and analysis are based on visual inspection of the dam by the performing organization.

Sturgeon Pool Dam was judged to be unsafe-non emergency due to a seriously inadequate spillway. Additional analysis and maintenance actions were recommended.

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SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

LOWER HUDSON RIVER BASIN

STURGEON POOL DAM

ULSTER COUNTY, NEW YORK
INVENTORY NUMBER NY 75

PHASE 1
INSPECTION REPORT
NATIONAL DAM
SAFETY PROGRAM

Prepared by

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Prepared For

DEPARTMENT OF THE ARMY
NEW YORK DISTRICT, CORPS OF ENGINEERS
NEW YORK, NEW YORK

TABLE OF CONTENTS

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam: Sturgeon Pool Dam

State Located: New York

County Located: Ulster

Stream: Wallkill River

Date of Inspection: August 29, 1978

ASSESSMENT

No conditions were noted during the inspection or from the review of available information which indicate that immediate emergency action is necessary to protect the safety of downstream residents. At the time of the inspection of Sturgeon Pool Dam the water level in the impoundment was 25 feet below the normal water level.

In 1978 the owner commissioned a consultant to evaluate the stability of the dam. The consultant recommended that additional investigations including coring of the concrete and foundation rock be completed, piezometers installed to monitor actual uplift, and further analysis of stability and stress be conducted.

The hydrologic analysis conducted for this report indicates that the spillway is adequate to pass the SPF (Standard Project Flood) with only 0.3 feet of freeboard remaining. The dam will be overtopped by approximately 6.4 feet during the PMF (Probable Maximum Flood) which is the recommended spillway design flood for this structure. As a result of our analysis we have rated the spillway as "seriously inadequate." In the interest of safety a detailed hydrologic and hydraulic study and construction of modifications should be completed in the near future to provide facilities capable of controlling the PMF.

At the present time the dam is undergoing some modifications. The ogee crest is being removed and replaced and the vertical joints are to be grouted to control seepage through the dam. Additional modifications and investigations are scheduled for late 1978 and 1979.

To ensure the future safety of this dam the following recommendations are made:

1. Conduct a more detailed hydrologic analysis to determine what modifications are necessary to increase spillway capacity to control the PMF.

2. Conduct the previously recommended drilling and testing from the lower gallery to determine the strength of the concrete, engineering properties of the foundation rock and actual uplift pressures on the dam.

3. Complete a structural and stability analysis of the dam reflecting the data obtained from the drilling and testing and considering the water level in the reservoir due to the Probable Maximum Flood (PMF).

4. Continue with plans to repair and maintain the structure including control of seepage and deterioration of the granite facing.

5. Implement a routine surveillence and maintenance program with written records kept.

6. Develop a formal warning and evacuation plan for downstream residents in the event of potential disaster due to a major storm or problems with the structure.

7. If the hydrologic and stability analyses indicate the dam can withstand overtopping due to the PMF modifications to the structure to prevent the abutments from erosion may be required.

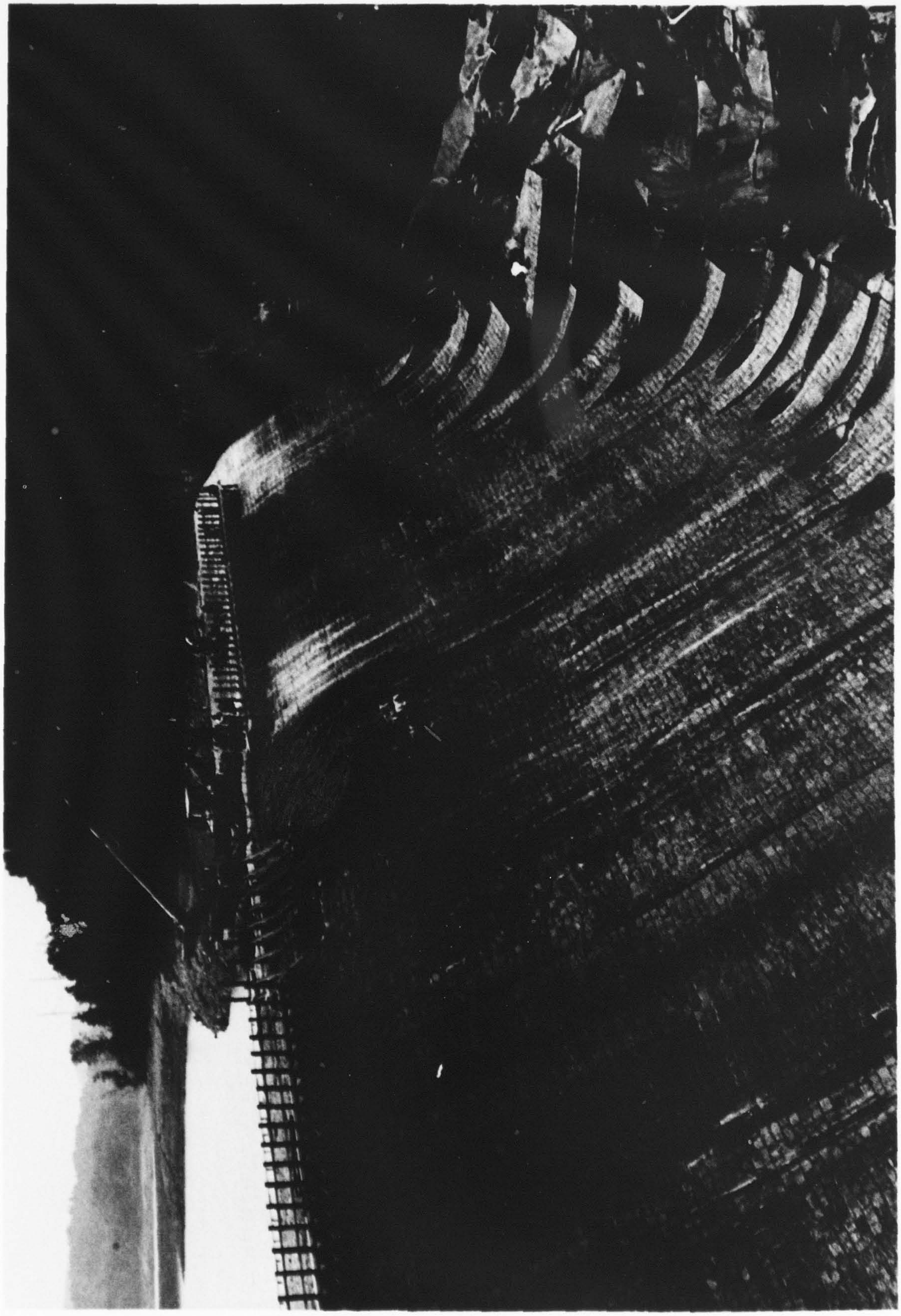
8. Presently the owner plans to plug the reservoir drainage facilities at the base of the dam. Consideration should be given to repair of the sluice gates and continued use of the facilities for reservoir drawdown. If the drainage chambers are plugged 70 feet of the reservoir will not be able to be drained.

Approved: R. Jeffrey Kimball

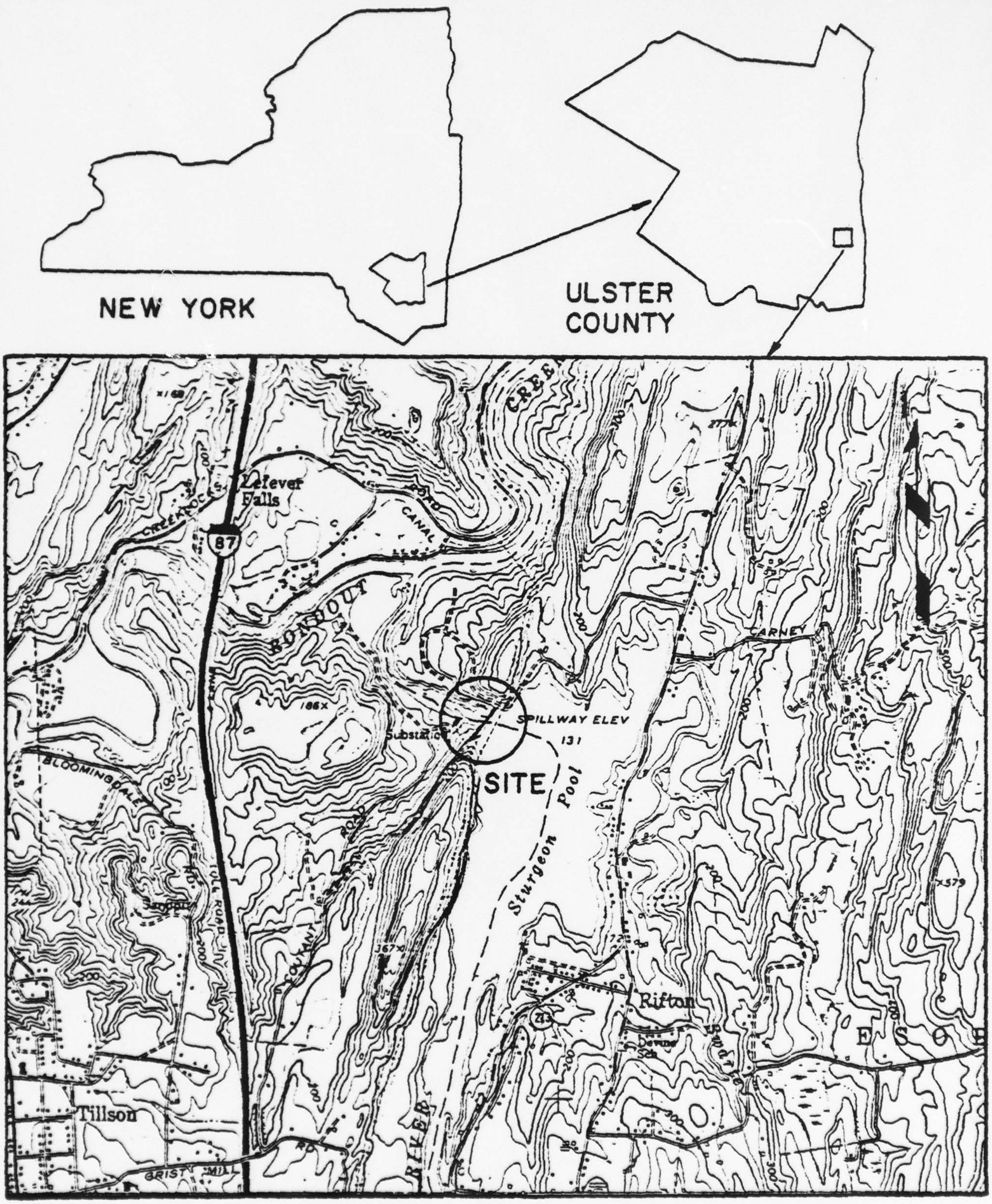
R. Jeffrey Kimball, P.E.
L. ROBERT KIMBALL & ASSOCIATES
Registration No. PA 26275E

Approved: Clark H. Benn

CLARK H. BENN
Colonel, Corps of Engineers
District Engineer
29 September 1978



OVERVIEW OF DOWNSTREAM
FROM RIGHT ABUTMENT



STURGEON POOL DAM

SITE LOCATION MAP

SCALE: 1"=2000'

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
STURGEON POOL DAM ID # 75

1.1 General:

a. Authority: Authority is provided by the National Dam Inspection Act Public Law 92-367.
Contract Number: DACW51-78-C-0025

b. Purpose of Project: Evaluation of non-Federal dams to identify dams which are a threat to life and property.

1.2 Description of Project:

a. Description of Dam and Appurtenances: Sturgeon Pool Dam is a concrete gravity dam constructed in 1922. The structure consists of a central overflow ogee section 490 feet long. The right abutment is a non-overflow bulkhead and the left abutment is a non-overflow section housing a high level overflow and the intake for the hydro power penstocks. The dam is 670 feet wide from abutment to abutment. The base of the dam is at elevation 20.0 feet, the crest of the overflow section at elevation 128.5 feet and the top of the non-overflow section at 141.5 feet. The maximum structural height is 108.5 feet. The dam was constructed with vertical contraction joints, spaced at 50 foot intervals. Essentially the structure is formed by 10 separate sections. The dam was faced with 8 inch to 16 inch granite blocks shortly after construction. In the recent past this facing has greatly deteriorated.

The central ogee section serves as the emergency spillway. Under normal conditions, flashboards are installed in the ogee section to raise the normal pool from the ogee crest elevation of 128.5 feet to elevation 131.0 feet at the center third of the ogee section and 132.0 feet at the outer thirds. It is reported by the owner's personnel that the flashboards are designed to fail at a reservoir water level of 132.5 feet. Flip buckets were constructed at the base of the downstream face near both ends of the ogee section to protect the foundation and abutments against erosion.

Six sluice gates are located at the centerline of the structure near the base (invert elevation 32.0 feet). The sluice gates control flow through six, 6 foot diameter concrete openings through the structure. The gates are opened by electrically driven (or manually operated) valves located in the lower passageway at the base of the dam. The sluice gates were last opened in 1962 or 1963 and because of difficulties with the gates they have not been opened since. At the present time the gates are considered to be inoperable as closure is doubtful.

The lower inspection gallery is connected to an upper inspection galley by a 6 foot x 6 foot vertical shaft with a steel spiral stairway. The upper inspection galley runs from the left bulkhead to the right bulkhead with access in both bulkheads. The elevation of the bottom of the upper inspection gallery is 110.0 feet.

As previously mentioned the left abutment bulkhead houses the three power penstocks and what was originally the high level overflow. The high level overflow was used for debris removal. During high discharges flow through the opening damaged the lawn behind the turbine house and the high level discharge has since been concreted shut.

The three power penstocks are 10 foot diameter steel conduits. These conduits are presently the only means for regulating the reservoir water level below the ogee crest. The invert of the penstock inlet in the bulkhead is at elevation 93.0 feet. The penstocks can drain the reservoir to elevation 103.0 feet according to the owner's personnel. However, discharges can be made only with the power turbines operating,as the turbine blades will be damaged if free discharge is allowed.

A track mounted crane and three large gates are located on top of the left abutment bulkhead. The gates can be lowered by tracks into the penstocks to allow the penstocks to be drained. Normally the penstocks are under pressure with discharge controlled by operation of the power turbines. A fourth gate is located on the left bulkhead for closure of the high level overflow previously described. The crane tracks were installed in the ogee crest to allow the crane to travel the crest for work on the inlet of the sluice gates. These tracks are being removed with the present repairs.

Since 1973 the structure has been the subject of several engineering studies. A discussion of past engineering evaluations is presented in Section 2: Engineering Data,with copies of the past reports presented in Appendix D.

As a result of past studies, the dam is presently undergoing considerable cosmetic repair. At the time of the inspection, a contractor was working on removal of the top of the ogee section to an approximate depth of 5 to 6 feet. The crest will be restored with higher strength concrete by the end of the summer. (See appendix E for construction drawings).

At the time of the inspection the water level in the impoundment was drawn down to elevation 108.0 feet to facilitate construction. As discussed in the reports in Appendix D, seepage has been noted in the upper gallery in the past. However, the water level was below the upper gallery at the time of the inspection. The seepage is apparently through the vertical joints. These joints will be grouted this summer.

The Dashville Dam, a smaller hydropower structure is located 2 miles upstream on the Wallkill River.

b. Location: The dam is located on the Wallkill River near the confluence with Rondout Creek. The dam is located near Rosendale, Ulster County, New York. The location can be found on the Rosendale, New York, 7.5 minute series quadrangle, (See site location map).

c. Size Classification: The structure is a high hazard potential dam with development along Rondout Creek downstream.

e. Ownership: Sturgeon Pool Dam is owned by Central Hudson Gas and Electric Corporation.

f. Purpose of Dam: The reservoir is operated and utilized for hydropower production.

g. Design and Construction History: The dam was designed in 1922 by the J.G. White Engineering Corporation of New York. The dam was constructed in 1922 by the Foundation Company of New York. Design drawings are available from the owner, however, no design report was available.

Several studies have been completed in the recent past for this structure. See Section 2 for discussion.

h. Normal Operating Procedures: Under normal conditions, the dam is operated at as high a reservoir level as possible to obtain maximum power production. The flashboards are usually in-place. The power station can produce 15 megawatts through three turbines (5 megawatts each). All normal discharges are presently through the power penstocks. The power units are operated as needed. The owner is not required to maintain a continuous flow from the structure.

1.3 Pertinent Data:

a. Drainage Area: The drainage area above the dam is approximately 787 square miles.

b. Discharge at Damsite:

Maximum Known Flood at Damsite: 1955-56 - maximum water elevation 136.0'

Spillway Capacity at Maximum Design Pool Elevation: 90,000 cfs from design drawings

Gated Spillway Capacity at Maximum Pool Elevation: Presently Inoperable

Ungated Spillway Capacity at Maximum Pool Elevation: 90,720 cfs

Normal Penstock Discharge Capacity: 18,000 cfs

c. Elevation: (feet above MSL)

Top of Dam: 141.5

Crest of Ogee Overflow Section: 128.5

Top of Flashboards: 131.0 at center third, 132.0 at outer thirds

Streambed at Centerline of Dam: Approximately 20.0

Normal Tailwater: 20.0

Maximum Design Pool Elevation: 141.3

Invert of Six Sluice Gates at Base of Dam: 32.0'

Invert of Penstock Inlets: 93.0'

d. Reservoir:

Length of normal pool: 11,000 feet

Length of Maximum Pool: 11,000 feet - Dashville Dam upstream

e. Storage: (acre-feet)

Normal pool: Approximately 8,000

Design Surcharge: 2,894

f. Surface Area: (acres)

Top of Dam: 339 at elevation 141.5'

Normal Pool: 212 at elevation 131.0'

g. Dam:

Type: Concrete Gravity

Length: 670 feet

Height: 108.5 feet

Top Width: Varies - minimum at ogee section 4'

Side Slopes: Upstream vertical
Downstream 0.67:1

Zoning: None

Impervious Core: None

Cutoff: Upstream toe into rock

Grout Curtain: None

h. Diversion and Regulating Facilities:

Type: Six 6' diameter openings

Length: 75.5' approximate

Closure: Electric motors or manual operation of values in lower observation gallery - presently not operable.

i. Spillway:

Type: Overflow section center of dam

Length: 490 feet

Crest Elevation: 128.5 feet ogee section

Gates: Flashboards installed to elevation 131.0 at center, elevation 132.0 end sections

Upstream Channel: None

Downstream Channel: Face of overflow section to Rondout Creek

j. Regulating Outlets:

Regulation of water level presently by discharge through power penstocks.

Maximum Discharge Capacity: 18,000 cfs (from owner)

Inlet Invert: 93.0 feet

Minimum Water Level at Which Penstocks will operate: 103.0 feet

SECTION 2: ENGINEERING DATA

2.1 Design: The structure was designed by J.G. White Engineering Corporation in 1922. Design drawings which include typical sections and details and a stress analysis for the dam are available. No design calculations are available.

2.2 Construction: Little information was available on the construction of the dam. The correspondence from the owner to the state during construction indicates that efforts were made to found the dam on competent rock.

2.3 Operation: Little operational data was available. Reservoir water level records are apparently maintained.

2.4 Recent Engineering Studies: The owner commissioned the Chas. T. Main Company of New York in 1973 to conduct a study of the alternatives necessary for retirement of the Sturgeon Pool Hydro Generating Plant. This report outlined the measures to be taken to abandoned the site. The analysis included evaluation of the hydrology and hydraulics of the dam and stability of the dam. The report implies that the dam was stable and not in need of repair to insure the safety of the structure.

Due to recent energy and economic conditions Central Hudson decided to keep the hydro power station in production. In March of 1978 Acres American Inc. of Buffalo, New York was contracted to evaluate the Sturgeon Pool Dam. The object of the evaluation was to:

"a. Undertake a structural inspection of the dam and a geological examination of the damsite area;"

"b. Perform a stability analysis of the dam;"

"c. Submit conclusions and recommendations to Central Hudson Gas and Electric regarding the condition of the dam and any on-going work which may be required."

Excerpts from the Summary of the Acres American report are presented below. The entire report is contained in Appendix D of this report with discussion of the stability analysis presented in Section 6.

ACRES AMERICAN 1978 REPORT - SUMMARY

" The geological examination indicated the dam is founded on interbedded, highly folded strata of greywacke, siltstone and shale. Overall, the rock mass appears to exhibit a fairly high compressive strength and is relatively impermeable. The stability of the rock mass is controlled by bedding planes in the strata and jointing in the greywacke."

"The structural examination did not indicate any obvious structural deficiencies. Seepage was observed at several points on the downstream face. The flow of water under pressure was observed on the dam at two locations near the right abutment."

"The dam was analyzed to determine its relative safety against overturning and sliding. The results of the analysis indicated that the overflow section of the dam has an adequate factor of safety against overturning under normal operating conditions, but the factor of safety against overturning during extreme flood conditions was slightly less than normally accepted limits. The factors of safety against sliding were found to be extremely sensitive to the parameters assigned to the foundation rock, and the stability against sliding cannot be confirmed until more definitive data regarding the shearing resistance of the foundation is obtained. The analysis indicated that reduction in hydrostatic uplift in the foundation rock by the use of pressure relief drains drilled into the foundation would be a significant beneficial effect on stability."

"The results of the concrete coring test program indicate the possibility that some zones of weak concrete exist in the dam, and stress levels in the structure may exceed allowable concrete strengths in these zones if they in fact do exist."

"CONCLUSIONS"

"-the stability of the dam with respect to sliding should be confirmed by field sampling and laboratory testing of the foundation rock to determine its overall frictional resistance"

"-subject to confirmation of adequate frictional resistance of the foundation rocks, the stability of the non-overflow and intake sections of the dam are acceptable"

"-all conditions of stability can be improved by installing pressure relief drain holes in the foundation"

"-the concrete coring and testing program should be extended to other areas of the dam in view of some of the poor quality core samples previously recovered"

" Recommendations For Futher Work"

"-install 4 or 5 NX size exploratory drill holes from the lower gallery and from the downstream toe of the dam into the rock foundation to examine the foundation strata and to provide rock cores for further testing. These holes should be of sufficient depth to determine rock stratigraphy accurately"

"-install piezometers in the drill holes to measure the actual hydrostatic uplift pressures on the base of the structure"

"-perform a limited laboratory test program to determine representative values of angle of shearing resistance (Ø) and cohesion (c) in order to confirm the sliding resistance of the dam foundation both on rock to rock and rock to concrete interfaces"

"-extend the concrete coring and testing program to the lower

gallery and at the toe of the dam to determine the compressive strength of the concrete near the base of the dam"

"-perform a more detailed analysis to determine the distribution of stress levels in the dam to compare with actual compressive strength test results obtained above"

"-areas of significant flow through construction joints in the dam or from the toe of the dam should be carefully mapped, and the practicality of sealing the construction joints by pressure grouting should be studied by attempting a field test. In areas where high flows are coming from behind the granite blocks near the face drain, holes should be drilled through the blocks. Sealing at the exit points of the flow should not be attempted"

"-the granite blocks on the downstream face of the overflow structure and in the crest should be carefully checked and reseated using mortar and epoxy resin where necessary to minimize further deterioration of the underlying concrete"

"-depending on the results of the drilling program to measure hydrostatic uplift on the base of the dam, and the results of the laboratory tests, a program should be formulated for drilling of vertical drain holes from the lower gallery of the dam to provide effective reduction in the uplift pressures. Normally these drain holes should be drilled into the rock approximately one half the height of the dam and spaced at 10 foot intervals. Near the ends of the gallery they should be fanned on the plane parallel to the longitudinal axis of the dam to effect coverage of the adjacent portions of the foundation'

"-the feasibility of replacing the areas of deteriorated granite blocks on the crest of the dam with a high strength concrete bonded to the underlying existing concrete should be reviewed"

A second study of the Sturgeon Pool Dam was completed in 1978 by the Chas T. Main Company. A draft copy of the Main report was available and is included in Appendix D. The report summarizes the repairs which are recommended for 1978 and 1979. Construction drawings for these repairs, prepared by C.T. Main were also available and are presented in Appendix E.

In summary the work recommended in the engineering reports which is currently being done or to be done in 1978 is listed below:

1. Removal of the crest of the overflow section of the dam until high strength concrete is encountered. Replace the crest with reinforced concrete and a concrete cap.

This removal work was in progress at the time of our inspection. The relative strength of concrete encountered during removal was being tested by a jack hammer. Samples of the exposed concrete will be tested for strength by laboratory testing. All work in progress was being supervised

by C.T. Main personnel.

2. During September of 1978, vertical drill holes (6" to 8" diameter) will be drilled at each of the construction joints by Layne New York. The holes will be drilled to a depth of 35' below the exposed crest surface. Each drill hole will be pressure grouted. The grout will be drilled out and an asphalt plug placed in each drill hole. This work is being done to reduce the seepage through the contraction joints.

Additional work to be completed will include: extension of horizontal drains and removal of vertical drains behind the granite facing, drilling of test holes into the foundation from the lower gallery to install piezometers and relief wells, repairs to reduce seepage around penstocks.

2.5 Evaluation: While little construction data is available, the recent studies provide sufficient data to evaluate the structure.

SECTION 3: VISUAL INSPECTION

3.1 Findings:

a. General: The Sturgeon Pool Dam was inspected by L. Robert Kimball and Associates personnel on August 29, 1978 accompanied by the owners personnel.

b. Dam: The inspection was conducted at a time when conditions were markedly different from normal operating conditions. The reservoir pool level was drawn down 25' below the normal pool to facilitate construction on the crest. Seepage through vertical contraction joints into the upper gallery and around the power penstocks noted during previous inspections was not occuring due to the low water level. Considerable work was being performed on the crest to repair damaged areas of the granite facing.

c. Appurtenant Structures: The valves for the six sluice gates were observed from the lower gallery. The valves are reported as inoperable. Seepage around the valves is apparent as water is discharging from two of the openings on the downstream face. The outside of the penstocks appeared to be in reasonably good condition. It is doubtful that the crane and gates at the left abutment section could be utilized to close off the penstock inlets.

d. Reservoir Area: The reservoir slopes range from gently sloping to steeply sloping. No signs of instability were noted. It is reported that the Wallkill River carries a considerable sediment load which creats a silt buildup in the reservoir.

e. Downstream Channel: Immediately downstream of the dam the Wallkill River joins Rondout Creek. At the dam the downstream channel slopes are steep and vegetated. No signs of instability were noted.

3.2 Evaluation: The visual inspection did not reveal any signs of instability or reasons for concern. As mentioned most of the major causes for concern noted during past inspections by the other engineers were not visable during our inspection due to the reduced water level.

SECTION 4: OPERATIONAL PROCEDURES

4.1 Procedures: The dam is operated as a hydro power facility. Ideally, a high water level is maintained in the reservoir. Normal discharge is through the power penstocks during power production. The emergency overflow is the only discharge facility presently functional.

To maintain a high water level flashboards are installed in the crest of the overflow sections. The flashboards are kept in place year round.

The power station is not operated continuously. Approximately one hour is needed to get to the site and complete start up operations after personnel are notified.

4.2 Maintenance of Dam: The dam is maintained by the owner. At present a major maintenance program is being conducted using contractors and is being supervised by the owner's consultant.

4.3 Maintenance of Operating Facilities: The operating facilities are in need of maintenance. The sluice gates are not operable and will probably be grouted shut in the near future. Some maintenance is required and has been scheduled for the intake of the penstocks.

4.4 Description of any Warning System in Effect: No warning system in effect.

4.5 Evaluation: Recent maintenance programs should render the structure and appurtences in good condition. A routine maintenance program has been recommended by the consultant and will apparently be followed.

SECTION 5: HYDRAULIC/HYDROLOGIC

5.1 Hydrologic Evaluation of Features:

a. Design Data: No hydrologic or hydraulic design data was available for the spillway, sluice gates or penstocks. Available construction drawings give the data necessary to develop hydraulic information.

The report prepared in 1973 by C.T. Main indicates that both SPF and PMF routings were conducted. However, only the SPF results were given in the report indicating a reservoir level of 137.3' with a peak discharge of 38,000 cfs.

The 1977 "Lower Hudson Hydrologic Flood Routing Model" indicates a peak SPF flood of 80,000 cfs for this area.

b. Experience Record: The maximum flood of record was in 1955 or 1956 when the reservoir water level reached elevation 136.0.

Past experience has indicated that discharges are experienced approximately 1 week per month from October to May over the overflow section.

c. Visual Observations: At the time of the inspection the overflow section was being repaired. The sluice gates are not operational and all discharges are through the penstocks.

d. Overtopping Analysis: Overtopping potential was investigated through the development of the probable maximum flood (PMF) for the watershed and subsequent routing of the PMF through the reservoir area. The PMF is that hypothetical flow induced by the most critical combination of precipitation, minimum infiltration losses, and concentration of run-off at a specific location, that is considered reasonably possible for a particular drainage area.

The drainage area contributing to Sturgeon Pool Dam is approximately 787 square miles. A report titled "Lower Hudson River Basin Hydrologic Flood Routing Model" prepared by Water Resources Engineer Inc. 1977, was reviewed. This report modeled several floods for the Wallkill River and Rondour Creek. This report was used to develop the hydrologic parameters and hydrographs for flood routing the SPF and PMF. The SPF hydrograph and unit hydrographs developed by this study were used as input data for the HEC-1 program to develop the inflow hydrograph to Sturgeon Pool.

Using Hydrometeorological Report No. 33, the PMP index rainfall was determined to be 21.0 inches for a 24 hour duration, 200 square mile basin. The percentages of the index rainfall applied to other durations were interpolated from the plot of drainage area versus percent of 24 hour, 200 square mile.

The ability of the Sturgeon Pool Dam to discharge the standard project flood (SPF) was evaluated. The inflow hydrograph for the standard project flood had a peak flow of 85,800 cfs. Routing through the impoundment did not reduce the peak discharge required.

The SPF outflow is indicative of a pool elevation of 141.2 feet above MSL leaving 0.3 feet of freeboard remaining.

As the results of the SPF routing indicated that the dam was nearly overtopped by the SPF a detailed routing of the PMF was not conducted. To estimate the reservoir level during the PMF the SPF flow was doubled. The corresponding water level considering flow over the non-overflow sections was then calculated assuming the peak inflow = peak outflow. Based on the analysis using the assumptions, the PMF flow will overtop the dam by 6.4 feet.

To allow the outflow hydrographs to be developed and routed several assumptions were made.

1. It was assumed that the flashboards were in place and that the reservoir water level was at elevation 131.0 when routing was initiated.

2. To calculate the discharge it was assumed that the flashboards would fail with a water level of 132.5'.

3. Storage data was calculated from information taken from the U.S.G.S. topographic quadrangle.

SUMMARY OF HYDROLOGIC ANALYSIS
STURGEON POOL

Elevation Top of Dam - 141.5

Elevation Crest of Spillway - 128.5 ogee crest

PMF ROUTING

PMF Peak : 171,600 cfs (estimated - 200% of SPF)

PMF After Routing through Reservoir: 171,600 cfs

Elevation of Routed PMF Corresponding to 171,600 cfs: 147.9

Dam Overtopped: 6.4'

Spillway Surcharge: 19.4'

SPF ROUTING

SPF Peak: 85,800 cfs

SPF After Routing through Reservoir: 85,800 cfs

Elevation of Routed SPF Corresponding to 85,800 cfs: 141.2

Freeboard Remaining: 0.3'

Spillway Surcharge: 12.7'

5.2 Evaluation of Hydrologic Analysis: The hydrologic analysis indicates that the structure can pass the SPF with only 0.3' of freeboard remaining. The crest is presently being modified. Should these modifications result in a slight decrease in hydraulic efficiency of the ogee crest the structure may not pass the SPF. According to the guidelines provided by the Corps of Engineers a large size, high hazard structure should be capable of passing the PMF. This structure cannot pass the PMF.

As a result of our analysis we have rated the spillway "seriously inadequate" as defined by the following Corps of Engineers definition;

"There is a seriously inadequate spillway if all of the following exist."

a. "There is high hazard to loss of life from large flows downstream of the dam."

b. "Dam failure resulting from overtopping would significantly increase the hazard to loss of life downstream from the dam from that which would exist just before overtopping failure."

c. "The dam and spillway are not capable of passing one-half of the probable maximum flood without overtopping failure."

The stability analyses conducted to date have not considered the water level due to the PMF or SPF or the affects on the structure due to overtopping. Therefore, failure due to overtopping is considered a possibility.

As modifications are presently being made to the overflow crest, the owner should make a detailed hydrologic and hydraulic study to determine what measures are necessary to increase spillway capacity. In the interest of safety, a detailed engineering study and necessary spillway modifications should be completed in the near future to provide facilities capable of controlling the PMF.

5.3 Hydraulic Evaluation of Flood Wave: A dam break analysis of the flood wave was computed for both total and partial failures.

The calculations indicate that for a partial breach the depth of water would be 22 feet a distance of 11,800 feet downstream. For a total failure the depth of water 11,800 feet downstream would be 38.5 feet.

Calculations of water depths at various distances downstream are included in Appendix B.

SECTION 6: STUCTURAL STABILITY

6.1 Evaluation of Structural Stability:

a. Visual Observations: At the time of the inspection the water level in the impoundment was being maintained at elevation 108, 25 feet below normal pool. No seepage through the dam or the abutments was noted. No signs of misalignment or instability were noted.

Past inspections by other engineers when the reservoir was at normal pool have noted seepage through the construction joints and into the upper gallery.

b. Design and Construction Data: No detailed design or construction data was available which would provide for complete analysis of structural stability. One design drawing was available which shows a summary of stress analyses in the dam. (See Appendix E). This data indicates that the resultant from maximum water level conditions lies slightly within the middle third of the section. This anlaysis used a maximum water elevation of 141.3'.*

As discussed in Section 2 several engineering studies have been made in the recent past. The 1973 study completed by Chas T. Main included an evaluation of structural stability. The study included an analyses of safety factors against overturning and sliding at the base. The results of the Main study are summarized below. The entire report is included in Appendix D. Two conditions were analyzed: normal water level and the expected SPF flood level (elevation 137.3').*

SUMMARY OF SAFETY FACTORS 1973 MAIN REPORT

	Safety Factor Against Overturing	Safety Factor Against Sliding
Normal Water level	1.28	7.5
SPF Water Level	1.09	6.7

Three sections were analyzed in the Main report. Only the results of the analysis of the ogee overflow section are presented as these were the lowest safety factors obtained.

The 1978 Acres American Report was concerned primarily with the stability of the structure. The analysis considered in the Acres American Report included sliding, buoyancy, overturning and overstressing. (See Appendix D for the entire report). The analyses included ice, silt and seismic loads and uplift pressures.

A summary of the Acres American Stability study is presented below. Again only the results of the analysis of the overflow section are presented. Both normal pool and flood conditions were analyzed (SPF projected at elevation 137.3').*

*Note: The SPF flood level calculated for this report is 141.2'.

SUMMARY OF SAFETY FACTORS 1978 ACRES AMERICAN REPORT OVERFLOW SECTION ONLY

		STATIC LOADING		DYNAMIC LOADING	
	Analysis	Normal Pool	Flood Pool	Normal Pool	Flood Pool
1.	Overturning	1.42	1.03	1.33	0.98
2.	Overturning considering 10' of under-cutting at downstream toe	1.33	0.98	1.24	0.96
3.	Buoyancy	2.60	1.40	2.06	1.27
		Safety Factors Required			
4.	Sliding	4.0	2.67	1.5	1.1

Assuming a Ø = 23°-25° the following values of cohesion for the rock were calculated as necessary to meet the desired S.F.

89 psi	34 psi	66 psi	29 psi

6. Assuming relief wells were installed to reduce uplift the following safety factors were calculated (a reduction of 1/3 of the head difference was assumed).,

6a.	Buoyancy	5.36	2.72	4.87	2.46
6b.	Overturning	2.18	1.57	1.96	1.46
6c.	Sliding required cohesion listed	82 psi	26 psi	60 psi	20 psi

In addition to the stability analyses Acres American also performed two dimensional stress analysis. The following excerpts are taken from Acres American's discussion of stress analysis for this dam.

"The maximum allowable coefficient of variation in concrete strengths of U.S. Bureau of Reclamation projects is 0.25; the value obtained for the concrete samples taken at Sturgeon Pool is 0.42."

"Criteria generally accepted by Bureau of Reclamation designers require that the strength of 80 percent of the test specimens

be greater than the design strength. Available test results show that 78 percent of the strength values for the Sturgeon Pool Dam samples are greater than 2500 psi, the average strength of the samples is 2757 psi, however, three of the samples feel apart or disintegrated either before or upon application of load."

"A tensile strength of 35 psi was found to exist at the base of the upstream face of the dam. The greatest compressive stress of 219 psi occures at the downstream toe of the dam."

"Normally, the above stresses would indicate an acceptable factor of safety; however, due to the poor test results from the concrete core samples, a dependable estimate of actual strength of the concrete is uncertain."

"Recognizing that the most critical stresses occur at the base of the dam, it is recommended that concrete cores be taken as close to the base section of the dam as possible, perhaps from the lower gallery. Appropriate tests should be performed in order to get specific information on the quality and strength of the concrete in this region of the dam. It is also advisable to do a more in-depth stress analysis of the dam in the form of a three-dimensional principal stress analysis or a finite element analysis."

Acres American also analyzed the foundation bearing capacity and found that the allowable bearing capacity of 1800 psi would be more than the maximum anticipated load of 50 psi.

c. Operational Data: The structure is operated with the flashboards in place and as high a water level as possible is maintained. This policy makes the dam susceptible to maximum loadings during major storms.

No other operational functions were noted which would adversely affect stability.

d. Post Construction Changes: The dam is presently undergoing some repairs which consist of replacing the ogee crest section. Additional studies and modifications are scheduled to repair the structure and increase the stability of the structure by possibly installing vertical relief drains.

c. Seismic Stability: As outlined in Section B, the seismic stability was evaluated by Acres American in 1978. This analysis utilized a coefficient of 0.06g.

The dam is located in seismic zone I with a recommended seismic load of 0.025g. Therefore, the seismic stability analyses are somewhat conservative.

It should be noted that a right lateral fault is located near the dam.

6.2 Stability Analysis Summary: As noted in section 6.1 b, several analyses have been conducted for this structure. However, only the original design evaluation considered a reservoir water level near the top of the dam. This analysis indicated the resultant was slightly inside the middle third. The assumptions made for this analysis are unknown.

The Acres American report concluded that additional drilling and testing of the concrete and rock foundation were necessary to vertify the dam stability. These additional studies have been scheduled.

A summary of Acres American conclusions and recommendations relative to stability is presented below.

"Computed factors of safety for overturning of the overflow section of the dam based on estimated uplift conditions fall slightly below normally accepted limits. This condition, however, can be significantly improved by installing pressure relief drain holes in the foundation of the dam. Sliding stability analysis of this section based on assumed rock parameters gives relatively low factors of safety. Testing of the foundation rock is required to permit the use of more accurate parameters in the analysis."

"Under extreme flood conditions, the factor of safety against overturning was computed to be 1.03, compared to the normal allowable value of 1.10."

"Slide stability is dependent on two rock strength parameters. In the absence of testing conservative estimates of these parameters must be used. It appears therefore, that the only practical means of resolving this aspect of representative samples of the foundation rock in direct shear."

"The results of the concrete coring and testing program indicated that some of the samples tested disintegrated before any measurable load could be applied. Extrapolating this failure rate to the more highly stressed lower area of the dam could result in zones where stress levels exceeded permissable concrete strengths."

"Our recommendations for further work are as follows:

"-Install 4 or 5 NX-size (3.0 inch) exploratory drill holes from the lower gallery and from the downstream toe of the dam into the rock foundation to examine the foundation strata and to provide rock cores for further testing. These holes should be of sufficient depth to give an adequate picture of the underlying of rock stratigraphy."

"-Install piezometers in the drill holes to measure the actual hydrostatic uplift pressures on the base of the structure;"

"-Perform a limited laboratory test program to determine representative values of angle of shearing resistance (Ø) and cohesion intercept (c) in order to confirm the sliding resistance of the dam foundation both on rock to rock and rock to concrete interfaces;"

"-Extend the concrete coring and testing program to the lower gallery and at the toe of the dam to determine the compressive strength of the concrete near the base of the dam;"

"-Perform a more detailed analysis to determine the distribution of stress levels in the dam to compare with actual compressive strength test results obtained above;"

"-Depending on the results of the drilling program to measure hydrostatic uplift on the base of the dam, and the results of the laboratory tests, a program should be formulated for drilling of drain holes from the lower gallery of the dam to provide effective reduction in the uplift pressure. Normally these drain holes should be drilled into the rock approximately one half the height of the dam and spaced at 10 foot intervals. Near the end of the gallery they should be fanned out with a 15 degree spacing from vertical to horizontal."

APPENDIX A

GEOLOGY

SECTION 7: ASSESSMENT/REMEDIAL MEASURES

7.1 Dam Assessment:

a. Safety: No conditions were noted during the inspection or preparation of this report which indicate immediate emergency action is necessary. However, past studies and the results of this investigation indicate that continued analysis and modifications are necessary to ensure future safety of the structure.

b. Adequacy of Information: The available information was adequate for this Phase I evaluation.

c. Urgency: At the present time modifications are being made to the dam crest. Also grouting of vertical joints is scheduled. Continued analyses have been recommended and should be implemented as soon as practical. The structure requires continued attention, however, no immediate emergency action is deemed necessary. Verification of structural stability and evaluation of modifications to the spillway to increase overflow capacity should be completed in the near future.

d. Necessity for Additional Work: More detailed structural and hydrologic analyses are recommended for this dam. See section 7.2 for recommendations.

7.2 Recommendations: To ensure the future safety of this dam the following recommendations are made.

1. Conduct a more detailed hydrologic analysis to determine what modifications are necessary to increase spillway capacity to control the PMF.

2. Conduct the drilling and testing from the lower gallery to determine the strength of the concrete, engineering properties of the foundation rock and actual uplift pressures on the dam.

3. Complete a structural and stability analysis of the dam reflecting the data obtained from the drilling and testing and considering the water level in the reservoir due to the Probable Maximum Flood (PMF).

4. Continue with plans to repair and maintain the structure including control of seepage and deterioration of the granite facing.

5. Implement a routine surveillence and maintenance program with written records kept.

6. Develop a formal warning and evaluation plan for downstream residents in the event of potential disaster due to a major storm or problems with the structure.

7. If the hydrologic and stability analyses indicate the dam can withstand overtopping due to the PMF, modifications to the structure to prevent the abutments from erosion may be required.

8. The sluice gates should be repaired and kept operational if practical.

STURGEON POOL DAM

Sturgeon Pool Dam is underlain by interbedded graywackes, siltstones and shales of the Normalskill Formation. The dam directly rests on the Austin Glen graywacke member of this formation which is usually less than three feet thick.

Members of the Normanskill Formation are of middle Ordovician Age and are associated with the Trenton Group. The depositional environment within the area of Sturgeon Pool Reservoir was part of a eugeosyncline during the Green Mointain Anticlinorium. Rapid deposition of sediments from this region caused westward emplacement of submarine gravity slides and turbidites. This created a mixture of muds and sands to be deposited. These sediments now form the graywackes of the Austin Glen and Normalskill. During periods of relatively stable sedimentation, shales and silts were deposited on the graywacks.

During the Taconic Orogeny, the bedrock surrounding Sturgeon Pool Reservoir was highly folded and faulted. According to the Acres report (May, 1978), there is a small fault with right laternal displacement near the right dam abutment. The fault is visible for only about 30 feet and goes beneath the masonry wall. This occurence was formed during the rock deformation associated with the Taconics. There is no evidence of recent movement and it appears to be stable.

APPENDIX B

HYDROLOGIC COMPUTATIONS

STURGEON POOL DAM

DRAINAGE AREA

FROM ENGINEER'S REPORT
AREA = 787 SQ. MI.

PRECIPITATION

FROM HYDROMETEOROLOGICAL REPORT 33,
PROBABLE MAXIMUM INDEX PRECIP. = 21.0"
(FOR 200 SQ. MI. – 24 HR.)

DEPTH–AREA–DURATION RELATIONSHIPS

		FOR SUBAREA 13 FROM MODEL	FOR NODE 109 AREA FROM MODEL
6-HR.	———	93 %	56 %
12-HR.	———	106 %	71 %
24-HR.	———	117 %	81 %
48-HR.	———	124 %	89 %

FROM EM 1110-2-1411,
STANDARD PROJECT INDEX PRECIP. = 10.0"
(FOR 200 SQ. MI. – 24 HR.)

[illegible]

FOR NODE 109 AREA THE SPF HYDROGRAPH IS GIVEN

SNYDER COEFFICIENTS

OBTAINED FOR NODE 109 RUNOFF CALCULATION OF THE PMF,
DRAINAGE AREA = 741.5 SQ. MI. (FROM MODEL)

LENGTH OF MAIN CHANNEL: L = 71.5 MILES

CENTROIDAL LENGTH OF BASIN: L_{CA} = 28.8 MILES

STURGEON POOL DAM

LAG TIME:

$$t_{pR} = C_t(.955)(L \times L_{CA})^{.3} + .25\,t_R$$

$$= 3.0(.955)(71.5 \times 28.8)^{.3} + .25(6)$$

$$= \underline{29.8 \text{ HR.}}$$

(C_t OBTAINED FROM MODEL; t_R FROM EM-1110-2-1411 PLATE #11)

UNIT HYDROGRAPH PEAK DISCHARGE:

$$Q_{pR} = \frac{640\,C_p\,A}{t_{pR}}$$

$$= \frac{(355)(741.5)}{29.8}$$

$$= \underline{8830 \text{ CFS}}$$

($640\,C_p$ OBTAINED FROM MODEL)

STURGEON POOL DAM

ELEVATION - DISCHARGE RELATIONSHIP

$$Q = 3.42 L H^{3/2}$$

SHARP CRESTED WIER

ELEV.	L	H	Q
131.0'	160'	0.0'	0 CFS
131.5'	160'	0.5'	193 CFS
132.0'	160'	1.0'	547 CFS
132.5'	160' +330'	1.5' 0.5'	1404 CFS

FOR C
REFER:
H-BOOK OF HYDRAULICS
(USBR TESTS)
BY BRATER & KING

$$Q = 3.95 L H^{3/2}$$

OGEE WEIR

ELEV.	L	H	Q
132.51	490'	4.01'	15,542 CFS
133.0	490'	4.5'	18,476 CFS
137.0	490'	8.5'	47,965 CFS
141.5	490'	13.0'	90,721 CFS

FOR C
REFER:

DESIGN OF SMALL
DAMS
BY U.S.B.R.

STURGEON POOL DAM

$Q_1 = 3.90\, L_1 H_1^{3/2}$ OGEE WEIR

$Q_2 = 2.88\, L_2 H_2^{3/2}$ BROAD-CRESTED WEIR

$L_1 = 490'$ $L_2 = 180'$

ELEV.	H_1	H_2	Q_1	Q_2	Q_{TOTAL}
142.0	13.5	0.5	94,790	183	94,973 CFS
150.0	21.5	8.5	190,510	12,847	203,357 CFS

STURGEON POOL DAM

ELEVATION - STORAGE RELATIONSHIP

ELEV. (FT.)	SURFACE AREA (ACRES)	Δ ELEV. (FT.)	TOTAL STORAGE (AC-FT)	TOTAL DISCHARGE (CFS)
131.0	212		0	0
		0.5		
131.5	218		108	193
		0.5		
132.0	224		218	547
		0.5		
132.5	230		332	1404
		0.01		
132.51	230		334	15,542
		0.49		
133.0	236		448	18,476
		4.0		
137.0	285		1490	47,965
		4.5		
141.5	339		2894	90,721
		0.5		
142.0	345		3065	94,973
		8.0		
150.0	441		6209	203,357

STURGEON POOL

HYDRAULIC EVALUATION OF FLOOD WAVE

$$Q_{MAX} = .29\sqrt{g}\, K^{.28}\, W_b D_b^{1.5}$$

$$S_i = \frac{12\, V_b}{Q_{MAX}} \qquad V_b = 10{,}900 \text{ A.F. @ TOP OF DAM}$$

$$\frac{\text{ATT. } Q_{MAX}}{Q_{MAX}} = \frac{0.91\, S_i}{S_i + T_s}$$

WHERE:

$$K = \frac{W_d}{W_b} \cdot \frac{Y_o}{D_b}$$

$$T_s = L\, t_s$$

$$t_s = \Delta S / \Delta Q$$

A) <u>FULL BREACH</u>

$$W_b = W_d = 670'$$

$$D_b = Y_o = 108.5'$$

$$Q_{MAX} = \underline{1{,}273{,}000 \text{ cfs}}$$

STURGEON POOL

		DIST. FROM DAM
REACH 1	L = 1650'	1650'

D_{DS} = 66' W = 1250' D_{AVE} = 80.2'

WATER SURFACE EL. 85.0

Q_{MAX} = 1,127,000 cfs

REACH 2	L = 2550'	4200'

D_{DS} = 66' W = 1070' D_{AVE} = 66'

WATER SURFACE EL. 83.0'

Q_{MAX} = 964,500 cfs

REACH 3	L = 2500'	6700'

D_{DS} = 66' W = 895' D_{AVE} = 66'

WATER SURFACE EL. 81.0'

Q_{MAX} = 806,700 cfs

REACH 4	L = 2350'	9050'

D_{DS} = 51.5' W = 1180' D_{AVE} = 56.3'

WATER SURFACE EL. 65.0'

Q_{MAX} = 733,100 cfs

STURGEON POOL

		Dist. from Dam
REACH 5	$L = 2750'$	11,800'

$D_{DS} = 38.5'$ $W = 1590'$ $D_{AVE} = 42.8'$

Water Surface El. 50.0

$Q_{MAX} = 638,500$ cfs

B) PARTIAL BREACH

$W_b = 150'$ $D_b = Y_0 = 108.5'$

$Q_{MAX} = 285,000$ cfs

REACH 1	$L = 1650'$	1650'

$D_{DS} = 34'$ $W = 790'$ $D_{AVE} = 58.8'$

Water Surface El. 53.0'

$Q_{MAX} = 263,300$ cfs

REACH 2	$L = 2550'$	4200'

$D_{DS} = 34'$ $W = 680'$ $D_{AVE} = 34'$

Water Surface El. 51.0'

$Q_{MAX} = 226,600$ cfs

STURGEON POOL

DIST. FROM DAM

REACH 3 $L = 2500'$ 6700'

$D_{DS} = 34'$ $W = 630'$ $D_{AVE} = 34'$

WATER SURFACE EL. 49.0'

$Q_{MAX} = 210{,}000$ cfs

REACH 4 $L = 2350'$ 9050'

$D_{DS} = 24.0'$ $W = 950'$ $D_{AVE} = 27.3'$

WATER SURFACE EL 37.5'

$Q_{MAX} = 193{,}700$ cfs

REACH 5 $L = 2750'$ 11,800'

$D_{DS} = 22.0'$ $W = 1050'$ $D_{AVE.} = 22.7'$

WATER SURFACE EL. 33.5'

$Q_{MAX} = 182{,}134$ cfs

****
HEC-1 VERSION DATED JAN 1973
UPDATED AUG 74
CHANGE NO. 01
****

STURGEON POOL DAM
RESERVOIR AT SPILLWAY ELEVATION
TEST SPF

JOB SPECIFICATION

NQ	NHR	NMIN	IDAY	IHR	IMIN	METRC	IPLT	IPRT	NSTAN
42	3	0	0	3	0	0	2	0	0

JOPER	NWT
3	0

********** ********** ********** ********** **********

SUB-AREA RUNOFF COMPUTATION

3-HOUR SPF HYDROGRAPH AT NODE 109

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
1	0	0	0	0	0	1

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
-1	0	741.10	0.0	0.0	0.0	0.0	0	0	0

INPUT HYDROGRAPH

1623.	1619.	1605.	1585.	1563.	1539.	1514.	1485.	1451.	1412.
1368.	1320.	1269.	1252.	1437.	1949.	2742.	3458.	3634.	3509.
5319.	13935.	30474.	52554.	71839.	79948.	76423.	67189.	58499.	51609.
46615.	43042.	40659.	39784.	39191.	38619.	37972.	37201.	36288.	35244.
34101.	32889.								

PEAK 6-HOUR 24-HOUR 72-HOUR TOTAL VOLUME

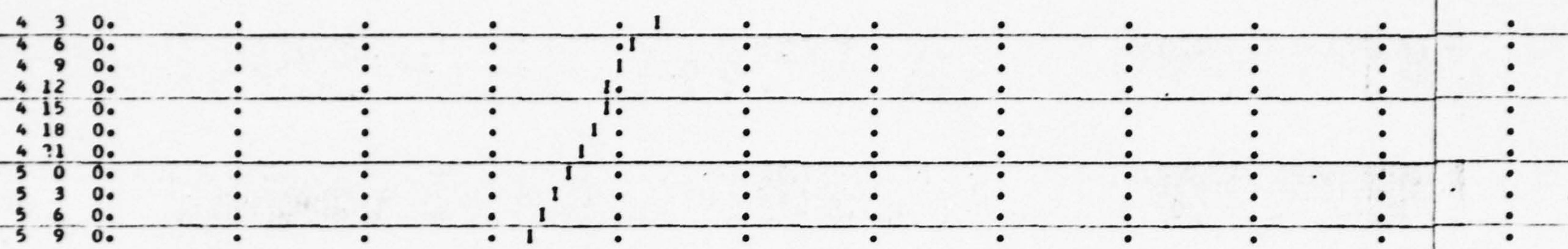
CFS	79948.	78186.	63085.	40689.	1006728.
INCHES		0.98	3.17	6.13	6.32
AC-FT		38790.	125191.	242242.	249731.

OVF

STATION 1

INFLOW I , OUTFLOW O AND OBSERVED FLOW *

```
        0.   10000.    20000.    30000.    40000.    50000.    60000.    70000.    80000.        0.        0.        0.
0  6  0. I       .         .         .         .         .         .         .         .         .         .         .
0  9  0. I       .         .         .         .         .         .         .         .         .         .         .
0 12  0. I       .         .         .         .         .         .         .         .         .         .         .
0 15  0. I       .         .         .         .         .         .         .         .         .         .         .
0 18  0. I       .         .         .         .         .         .         .         .         .         .         .
0 21  0. I       .         .         .         .         .         .         .         .         .         .         .
1  0  0. I       .         .         .         .         .         .         .         .         .         .         .
1  3  0.I        .         .         .         .         .         .         .         .         .         .         .
1  6  0.I        .         .         .         .         .         .         .         .         .         .         .
1  9  0.I        .         .         .         .         .         .         .         .         .         .         .
1 12  0.I        .         .         .         .         .         .         .         .         .         .         .
1 15  0.I        .         .         .         .         .         .         .         .         .         .         .
1 18  0.I        .         .         .         .         .         .         .         .         .         .         .
1 21  0.I        .         .         .         .         .         .         .         .         .         .         .
2  0  0.I        .         .         .         .         .         .         .         .         .         .         .
2  3  0. I       .         .         .         .         .         .         .         .         .         .         .
2  6  0.  I      .         .         .         .         .         .         .         .         .         .         .
2  9  0.  I      .         .         .         .         .         .         .         .         .         .         .
2 12  0.   I     .         .         .         .         .         .         .         .         .         .         .
2 15  0.   I     .         .         .         .         .         .         .         .         .         .         .
2 18  0.    I    .         .         .         .         .         .         .         .         .         .         .
? 21  0.         .   I     .         .         .         .         .         .         .         .         .         .
3  0  0.         .         .         I         .         .         .         .         .         .         .         .
3  3  0.         .         .         .         .         .  I      .         .         .         .         .         .
3  6  0.         .         .         .         .         .         .         . I       .         .         .         .
3  9  0.         .         .         .         .         .         .         .         I         .         .         .
3 12  0.         .         .         .         .         .         .         .     I   .         .         .         .
3 15  0.         .         .         .         .         .         .      I  .         .         .         .         .
3 18  0.         .         .         .         .         .       I .         .         .         .         .         .
3 21  0.         .         .         .         .         . I       .         .         .         .         .         .
4  0  0.         .         .         .         .      I  .         .         .         .         .         .         .
```

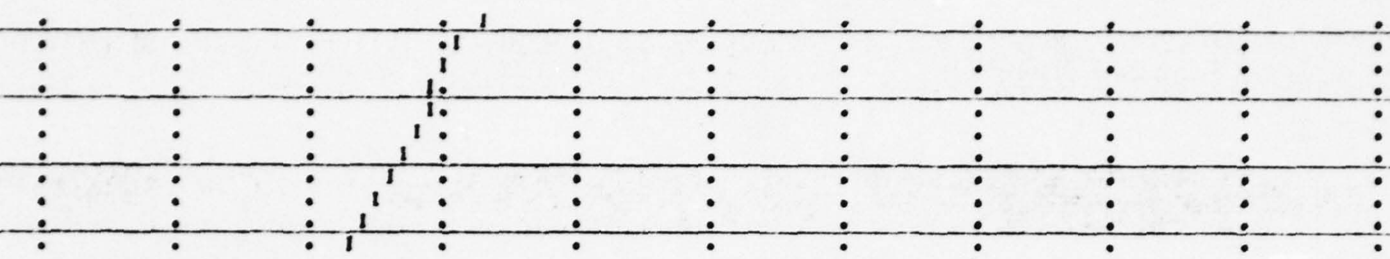


4 3 0.
4 6 0.
4 9 0.
4 12 0.
4 15 0.
4 18 0.
4 21 0.
5 0 0.
5 3 0.
5 6 0.
5 9 0.

```
*OVN*

      **********      **********      **********      **********      **********

                                   HYDROGRAPH ROUTING
                 ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME
                     2       1       0       0       0       0       0
                                      ROUTING DATA
                         QLOSS   CLOSS     AVG    IRES   ISAME
                           0.0     0.0     0.0       0       0

                 NSTPS   NSTDL     LAG   AMSKK       X     TSK   STORA
                     1       0       0     0.0     0.0     0.0      0.

                                  ROUTED FLOWS AT      2
     1623.     1619.     1605.     1585.     1563.     1539.     1514.     1485.     1451.     1412.
     1368.     1320.     1269.     1252.     1437.     1949.     2742.     3458.     3634.     3509.
     5319.    13935.    30474.    52554.    71839.    79948.    76423.    67189.    58499.    51609.
    46615.    43042.    40659.    39784.    39191.    38619.    37972.    37201.    36288.    35244.
    34101.    32889.

                          PEAK    6-HOUR   24-HOUR   72-HOUR   TOTAL VOLUME
                 CFS    79948.    78186.    63085.    40689.       1006728.
              INCHES                0.98      3.17      6.13           6.32
               AC-FT               38790.   125191.   242242.        249731.
```

```
*OVF*
                                                 STATION     2

                            INFLOW I , OUTFLOW O  AND OBSERVED FLOW *
        0.    10000.    20000.    30000.    40000.    50000.    60000.    70000.    80000.        0.        0.        0.
  0  6  0. I       .         .         .         .         .         .         .         .         .         .         .
  0  9  0. I       .         .         .         .         .         .         .         .         .         .         .
  0 12  0. I       .         .         .         .         .         .         .         .         .         .         .
  0 15  0. I       .         .         .         .         .         .         .         .         .         .         .
  0 18  0. I       .         .         .         .         .         .         .         .         .         .         .
  0 21  0. I       .         .         .         .         .         .         .         .         .         .         .
  1  0  0. I       .         .         .         .         .         .         .         .         .         .         .
  1  3  0.I        .         .         .         .         .         .         .         .         .         .         .
  1  6  0.I        .         .         .         .         .         .         .         .         .         .         .
  1  9  0.I        .         .         .         .         .         .         .         .         .         .         .
  1 12  0.I        .         .         .         .         .         .         .         .         .         .         .
  1 15  0.I        .         .         .         .         .         .         .         .         .         .         .
  1 18  0.I        .         .         .         .         .         .         .         .         .         .         .
  1 21  0.I        .         .         .         .         .         .         .         .         .         .         .
  2  0  0.I        .         .         .         .         .         .         .         .         .         .         .
  2  3  0. I       .         .         .         .         .         .         .         .         .         .         .
  2  6  0.  I      .         .         .         .         .         .         .         .         .         .         .
  2  9  0.  I      .         .         .         .         .         .         .         .         .         .         .
  2 12  0.   I     .         .         .         .         .         .         .         .         .         .         .
  2 15  0.   I     .         .         .         .         .         .         .         .         .         .         .
  2 18  0.    I    .         .         .         .         .         .         .         .         .         .         .
  2 21  0.         .   I     .         .         .         .         .         .         .         .         .         .
  3  0  0.         .         .         I         .         .         .         .         .         .         .         .
  3  3  0.         .         .         .         .         .  I      .         .         .         .         .         .
  3  6  0.         .         .         .         .         .         .         . I       .         .         .         .
  3  9  0.         .         .         .         .         .         .         .         I         .         .         .
  3 12  0.         .         .         .         .         .         .         .     I   .         .         .         .
  3 15  0.         .         .         .         .         .         .      I  .         .         .         .         .
  3 18  0.         .         .         .         .         .       I .         .         .         .         .         .
  3 21  0.         .         .         .         .         . I       .         .         .         .         .         .
  4  0  0.         .         .         .         .      I  .         .         .         .         .         .         .
```

4	3	0.
4	6	0.
4	9	0.
4	12	0.
4	15	0.
4	18	0.
4	21	0.
5	0	0.
5	3	0.
5	6	0.
5	9	0.



```
*OVN*

          **********          **********          **********          **********          **********

                                          SUB-AREA RUNOFF COMPUTATION

                 3-HOUR UNIT HYDROGRAPH AT SUBAREA 13
                              ISTAQ    ICOMP    IECON    ITAPE     JPLT     JPRT    INAME
                                  2        0        0        0        0        0        1

                                           HYDROGRAPH DATA
                IHYDG     IUHG    TAREA     SNAP    TRSDA    TRSPC    RATIO    ISNOW    ISAME    LOCAL
                    1       -1    46.00      0.0    46.00      0.0      0.0        0        0        0

                                             PRECIP DATA
                         SPFE      PMS       R6      R12      R24      R48      R72      R96
                        10.00      0.0      0.0      0.0      0.0      0.0      0.0      0.0
TRSPC COMPUTED BY THE PROGRAM IS 0.847

                                              LOSS DATA
               STRKR    DLTKR    RTIOL    ERAIN    STRKS    RTIOK    STRTL    CNSTL    ALSMX    RTIMP
                 0.0      0.0     1.00      0.0      0.0     1.00     1.50     0.15      0.0     0.01

                                     GIVEN UNIT GRAPH, NUHGQ   15
          1061.     2352.     2152.     1437.      960.      641.      428.      286.      191.      127.
            85.       57.       38.       25.       17.
                         UNIT GRAPH TOTALS    9857. CFS OR 1.00 INCHES OVER THE AREA

                                            RECESSION DATA
                              STRTQ     46.00     QRCSN     -0.35     RTIOR   3.00

                                           END-OF-PERIOD FLOW
                                     TIME     RAIN     EXCS     COMP Q
                                  0  6  0     0.00     0.00        41.
                                  0  9  0     0.00     0.00        37.
```

0	12	0	0.02	0.00	33.
0	15	0	0.02	0.00	30.
0	18	0	0.08	0.00	28.
0	21	0	0.16	0.00	28.
1	0	0	0.01	0.00	27.
1	3	0	0.01	0.00	24.
1	6	0	0.02	0.00	21.
1	9	0	0.02	0.00	19.
1	12	0	0.07	0.00	17.
1	15	0	0.07	0.00	17.
1	18	0	0.34	0.00	19.
1	21	0	0.70	0.02	38.
2	0	0	0.04	0.00	55.
2	3	0	0.04	0.00	49.
2	6	0	0.15	0.00	37.
2	9	0	0.15	0.00	31.
2	12	0	0.53	0.09	118.
2	15	0	0.53	0.09	317.
2	18	0	2.49	2.05	2579.
2	21	0	5.06	4.62	10037.
3	0	0	0.31	0.00	15483.
3	3	0	0.31	0.00	13032.
3	6	0	0.01	0.00	8711.
3	9	0	0.01	0.00	5821.
3	12	0	0.03	0.00	4973.
3	15	0	0.03	0.00	4455.
3	18	0	0.13	0.00	3992.
3	21	0	0.27	0.00	3576.
4	0	0	0.02	0.00	3204.
4	3	0	0.02	0.00	2871.
4	6	0	0.0	0.0	2572.
4	9	0	0.0	0.0	2305.
4	12	0	0.0	0.0	2065.
4	15	0	0.0	0.0	1850.
4	18	0	0.0	0.0	1658.
4	21	0	0.0	0.0	1485.

5	0	0	0.0	0.0	1331.
5	3	0	0.0	0.0	1192.
5	6	0	0.0	0.0	1068.
5	9	0	0.0	0.0	957.
		SUM	11.65	6.87	96203.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	15483.	14258.	8313.	3985.	96203.
INCHES		2.88	6.72	9.67	9.73
AC-FT		7074.	16497.	23727.	23864.

```
*OVF*
                                                    STATION      2

                                   INFLOW I , OUTFLOW O  AND OBSERVED FLOW *
      0.     2000.     4000.     6000.     8000.    10000.    12000.    14000.    16000.        0.        0.        0.
                                                                                                 PRECIP L   AND EXC
      0.        0.        0.        0.        0.        0.        0.        0.        0.        6.        4.        2.
0  6  0I         .         .         .         .         .         .         .         .         .         .         .
0  9  0I         .         .         .         .         .         .         .         .         .         .         .
0 12  0I         .         .         .         .         .         .         .         .         .         .         .
0 15  0I         .         .         .         .         .         .         .         .         .         .         .
0 18  0I         .         .         .         .         .         .         .         .         .         .         .
0 21  0I         .         .         .         .         .         .         .         .         .         .         .
1  0  0I         .         .         .         .         .         .         .         .         .         .         .
1  3  0I         .         .         .         .         .         .         .         .         .         .         .
1  6  0I         .         .         .         .         .         .         .         .         .         .         .
1  9  0I         .         .         .         .         .         .         .         .         .         .         .
1 12  0I         .         .         .         .         .         .         .         .         .         .         .
1 15  0I         .         .         .         .         .         .         .         .         .         .         .
1 18  0I         .         .         .         .         .         .         .         .         .         .         .
1 21  0I         .         .         .         .         .         .         .         .         .         .         .
2  0  0I         .         .         .         .         .         .         .         .         .         .         .
2  3  0I         .         .         .         .         .         .         .         .         .         .         .
2  6  0I         .         .         .         .         .         .         .         .         .         .         .
2  9  0I         .         .         .         .         .         .         .         .         .         .         .
2 12  0.I        .         .         .         .         .         .         .         .         .         .         .
2 15  0. I       .         .         .         .         .         .         .         .         .         .         .
2 18  0.         .  I      .         .         .         .         .         .         .         .         .       LLX
2 21  0.         .         .         .         .         I         .         .         .         .    LLXXXXXXXXXXXXXX
3  0  0.         .         .         .         .         .         .      I  .         .         .         .         .
3  3  0.         .         .         .         .         .    I    .         .         .         .         .         .
3  6  0.         .         .         .         .   I     .         .         .         .         .         .         .
3  9  0.         .        I.         .         .         .         .         .         .         .         .         .
3 12  0.         .    I    .         .         .         .         .         .         .         .         .         .
3 15  0.         . I       .         .         .         .         .         .         .         .         .         .
3 18  0.         .         I         .         .         .         .         .         .         .         .         .
```

```
3 21  0.         .       I .       .       .       .       .       .       .       .       .       .
4  0  0.         .      I  .       .       .       .       .       .       .       .       .       .
4  3  0.         .    I    .       .       .       .       .       .       .       .       .       .
4  6  0.         .   I     .       .       .       .       .       .       .       .       .       .
4  9  0.         .  I      .       .       .       .       .       .       .       .       .       .
4 12  0.         I         .       .       .       .       .       .       .       .       .       .
4 15  0.        I.         .       .       .       .       .       .       .       .       .       .
4 18  0.       I .         .       .       .       .       .       .       .       .       .       .
4 21  0.      I  .         .       .       .       .       .       .       .       .       .       .
5  0  0.      I  .         .       .       .       .       .       .       .       .       .       .
5  3  0.     I   .         .       .       .       .       .       .       .       .       .       .
5  6  0.    I    .         .       .       .       .       .       .       .       .       .       .
5  9  0.    I    .         .       .       .       .       .       .       .       .       .       .
```

OVN

********** ********** ********** ********** **********

COMBINE HYDROGRAPHS

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
2	2	0	0	0	0	0

SUM OF 2 HYDROGRAPHS AT 2

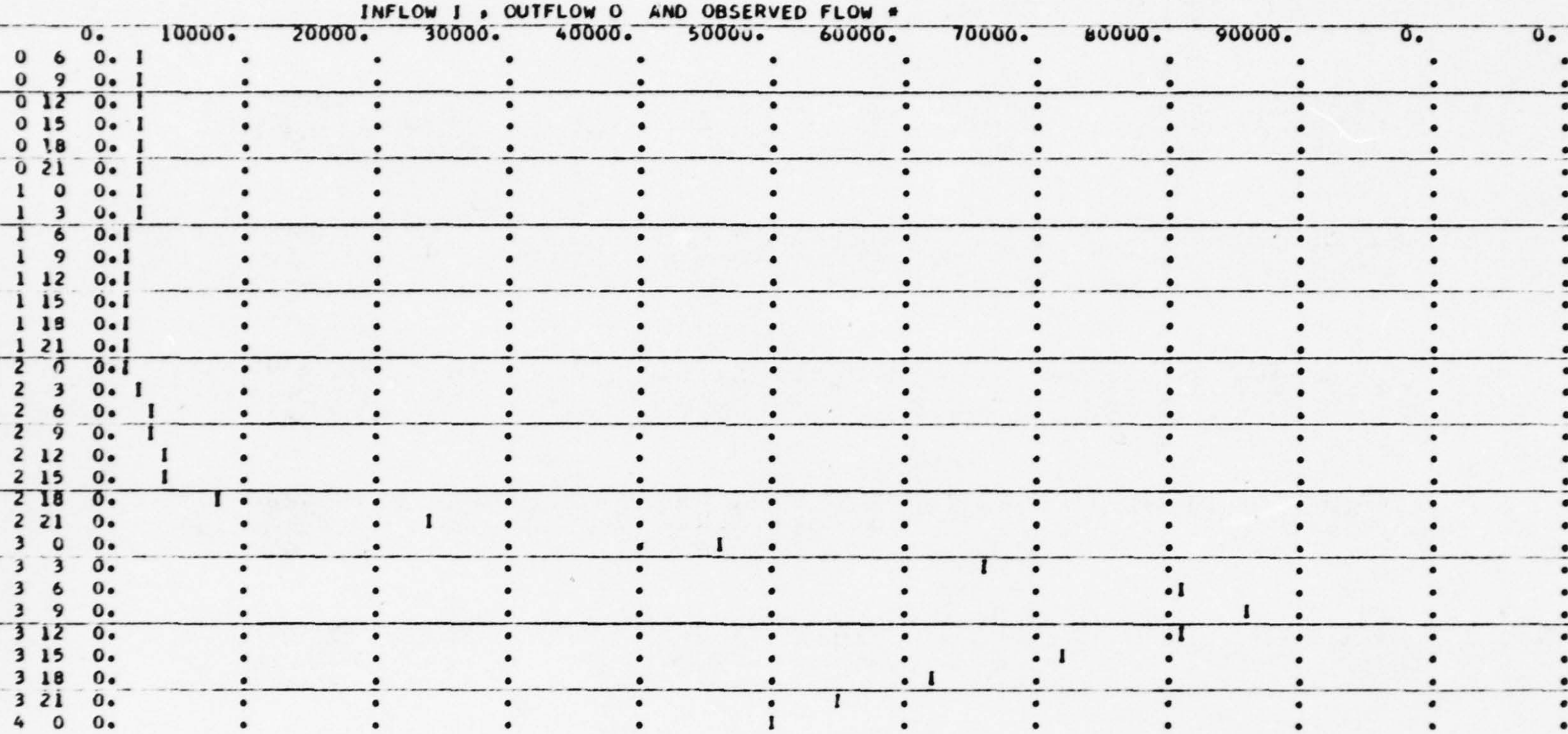
1664.	1656.	1638.	1615.	1591.	1567.	1541.	1509.	1472.	1431.
1385.	1337.	1288.	1290.	1492.	1998.	2779.	3489.	3752.	3826.
7898.	23972.	45957.	65586.	80550.	85769.	81396.	71644.	62491.	55185.
49819.	45913.	43231.	42089.	41256.	40469.	39630.	38686.	37619.	36436.
35169.	33846.								

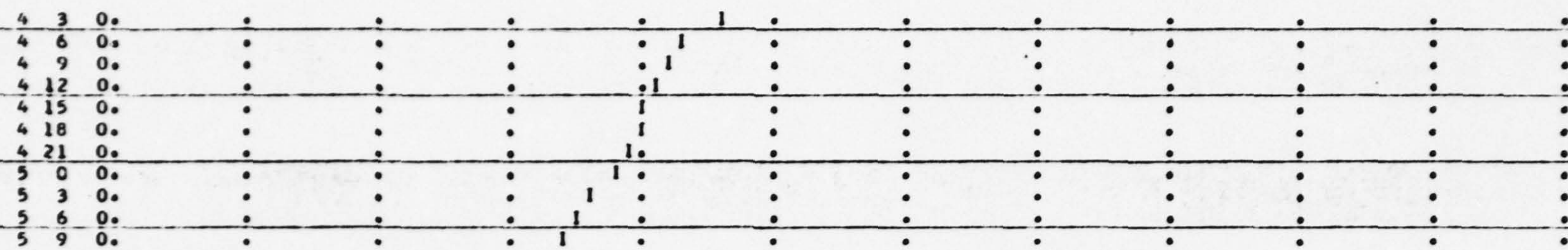
	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	85769.	83582.	69055.	44674.	1102930.
INCHES		0.99	3.26	6.34	6.52
AC-FT		41467.	137039.	265968.	273595.

OVF

STATION 2

INFLOW I , OUTFLOW O AND OBSERVED FLOW *




4 3 0.
4 6 0.
4 9 0.
4 12 0.
4 15 0.
4 18 0.
4 21 0.
5 0 0.
5 3 0.
5 6 0.
5 9 0.
I

```
*OVN*

          **********          **********          **********          **********          **********

                                                HYDROGRAPH ROUTING

                        RESERVOIR ROUTING
                                ISTAQ    ICOMP    IECON    ITAPE     JPLT     JPRT    INAME
                                    3        1        0        0        0        0        1
                                                     ROUTING DATA
                                         QLOSS    CLOSS      AVG     IRES    ISAME
                                           0.0      0.0      0.0        1        0

                                NSTPS    NSTDL      LAG    AMSKK        X      TSK    STORA
                                    1        0        0      0.0      0.0      0.0      -1.

      STORAGE         0.       108.       218.       332.       334.       448.      1490.      2894.      3065.      6209.
      OUTFLOW         0.       193.       547.      1404.     15542.     18476.     47965.     90721.     94973.    203357.
```

```
                                   TIME  EOP STOR    AVG IN   EOP OUT
                                 0  6  0     332.     1664.     1664.
                                 0  9  0     332.     1660.     1656.
                                 0 12  0     332.     1647.     1638.
                                 0 15  0     332.     1627.     1615.
                                 0 18  0     332.     1603.     1591.
                                 0 21  0     332.     1579.     1567.
                                 1  0  0     332.     1554.     1541.
                                 1  3  0     332.     1525.     1509.
                                 1  6  0     332.     1491.     1472.
                                 1  9  0     332.     1451.     1431.
                                 1 12  0     331.     1408.     1395.
                                 1 15  0     326.     1361.     1362.
                                 1 18  0     320.     1312.     1314.
                                 1 21  0     317.     1289.     1290.
                                 2  0  0     330.     1391.     1387.
```

2	3	0	332.	1745.	2084.
2	6	0	332.	2389.	2692.
2	9	0	332.	3134.	3575.
2	12	0	332.	3620.	3666.
2	15	0	332.	3789.	3912.
2	18	0	333.	5862.	7808.
2	21	0	576.	15935.	22100.
3	0	0	1284.	34964.	42122.
3	3	0	2004.	55772.	63611.
3	6	0	2495.	73068.	78564.
3	9	0	2733.	83160.	85831.
3	12	0	2617.	83582.	82276.
3	15	0	2318.	76520.	73175.
3	18	0	2001.	67067.	63518.
3	21	0	1758.	58838.	56118.
4	0	0	1570.	52502.	50401.
4	3	0	1435.	47866.	46418.
4	6	0	1334.	44572.	43545.
4	9	0	1285.	42660.	42167.
4	12	0	1258.	41672.	41397.
4	15	0	1229.	40862.	40565.
4	18	0	1200.	40049.	39762.
4	21	0	1167.	39158.	38821.
5	0	0	1130.	38152.	37780.
5	3	0	1089.	37027.	36609.
5	6	0	1044.	35803.	35354.
5	9	0	998.	34508.	34036.
		SUM			1100340.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	85831.	84053.	69187.	44565.	1100340.
INCHES		0.99	3.27	6.32	6.50
AC-FT		41701.	137300.	265316.	272952.

```
*OVF*
                                                       STATION     3

                                  INFLOW I , OUTFLOW O  AND OBSERVED FLOW *
           0.     10000.     20000.     30000.     40000.     50000.     60000.     70000.     80000.     90000.          0.          0.
0  6   0. I          .          .          .          .          .          .          .          .          .          .          .
0  9   0. I          .          .          .          .          .          .          .          .          .          .          .
0 12   0. I          .          .          .          .          .          .          .          .          .          .          .
0 15   0. I          .          .          .          .          .          .          .          .          .          .          .
0 18   0. I          .          .          .          .          .          .          .          .          .          .          .
0 21   0. I          .          .          .          .          .          .          .          .          .          .          .
1  0   0. I          .          .          .          .          .          .          .          .          .          .          .
1  3   0. I          .          .          .          .          .          .          .          .          .          .          .
1  6   0.I           .          .          .          .          .          .          .          .          .          .          .
1  9   0.I           .          .          .          .          .          .          .          .          .          .          .
1 12   0.I           .          .          .          .          .          .          .          .          .          .          .
1 15   0.I           .          .          .          .          .          .          .          .          .          .          .
1 18   0.I           .          .          .          .          .          .          .          .          .          .          .
1 21   0.I           .          .          .          .          .          .          .          .          .          .          .
2  0   0.I           .          .          .          .          .          .          .          .          .          .          .
2  3   0. I          .          .          .          .          .          .          .          .          .          .          .
2  6   0. IO         .          .          .          .          .          .          .          .          .          .          .
2  9   0.  IO        .          .          .          .          .          .          .          .          .          .          .
2 12   0.   I        .          .          .          .          .          .          .          .          .          .          .
2 15   0.   I        .          .          .          .          .          .          .          .          .          .          .
2 18   0.     I O    .          .          .          .          .          .          .          .          .          .          .
2 21   0.            .      I   .  O       .          .          .          .          .          .          .          .          .
3  0   0.            .          .          .     I    . O        .          .          .          .          .          .          .
3  3   0.            .          .          .          .          .      I   .        O .          .          .          .          .
3  6   0.            .          .          .          .          .          .          .  I       O.          .          .          .
3  9   0.            .          .          .          .          .          .          .          .  I  O    .          .          .
3 12   0.            .          .          .          .          .          .          .          . O I      .          .          .
3 15   0.            .          .          .          .          .          .          .  O    I  .          .          .          .
3 18   0.            .          .          .          .          .          .        O   I       .          .          .          .
3 21   0.            .          .          .          .          .      O   I.         .          .          .          .          .
4  0   0.            .          .          .          .          O  I       .          .          .          .          .          .
```

```
4  3   0.            .           .           .           .      0 1  .           .           .           .           .           .           .
4  6   0.            .           .           .           .   01      .           .           .           .           .           .           .
4  9   0.            .           .           .           . 01        .           .           .           .           .           .           .
4 12   0.            .           .           .           .01         .           .           .           .           .           .           .
4 15   0.            .           .           .           .1          .           .           .           .           .           .           .
4 18   0.            .           .           .           1           .           .           .           .           .           .           .
4 21   0.            .           .           .          1.           .           .           .           .           .           .           .
5  0   0.            .           .           .         1 .           .           .           .           .           .           .           .
5  3   0.            .           .           .       1   .           .           .           .           .           .           .           .
5  6   0.            .           .           .     01    .           .           .           .           .           .           .           .
5  9   0.            .           .           .    01     .           .           .           .           .           .           .           .
```

********** ********** ********** ********** **********

NAO

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	1	79948.	78186.	63085.	40689.	741.10
ROUTED TO	2	79948.	78186.	63085.	40689.	741.10
HYDROGRAPH AT	2	15483.	14258.	8313.	3985.	46.00
2 COMBINED	2	85769.	83582.	69055.	44674.	787.10
ROUTED TO	3	85831.	84053.	69187.	44565.	787.10

APPENDIX C

PHOTOGRAPHS

Photograph Index

1. View of downstream slope and right abutment. Note: leekage through sluice gate.

2. Upstream face. Note: crest removal.

3. Intakes for penstocks.

4. Crane and bulkheads for use when repairing penstocks.

5. Downstream view of left abutment and penstocks.

6. Discharge end of powerhouse.

7. Contractor removing spillway crest. Note: granite facing with deteriorated concrete.

8. Spillway crest removed near right abutment.

9. Downstream channel.

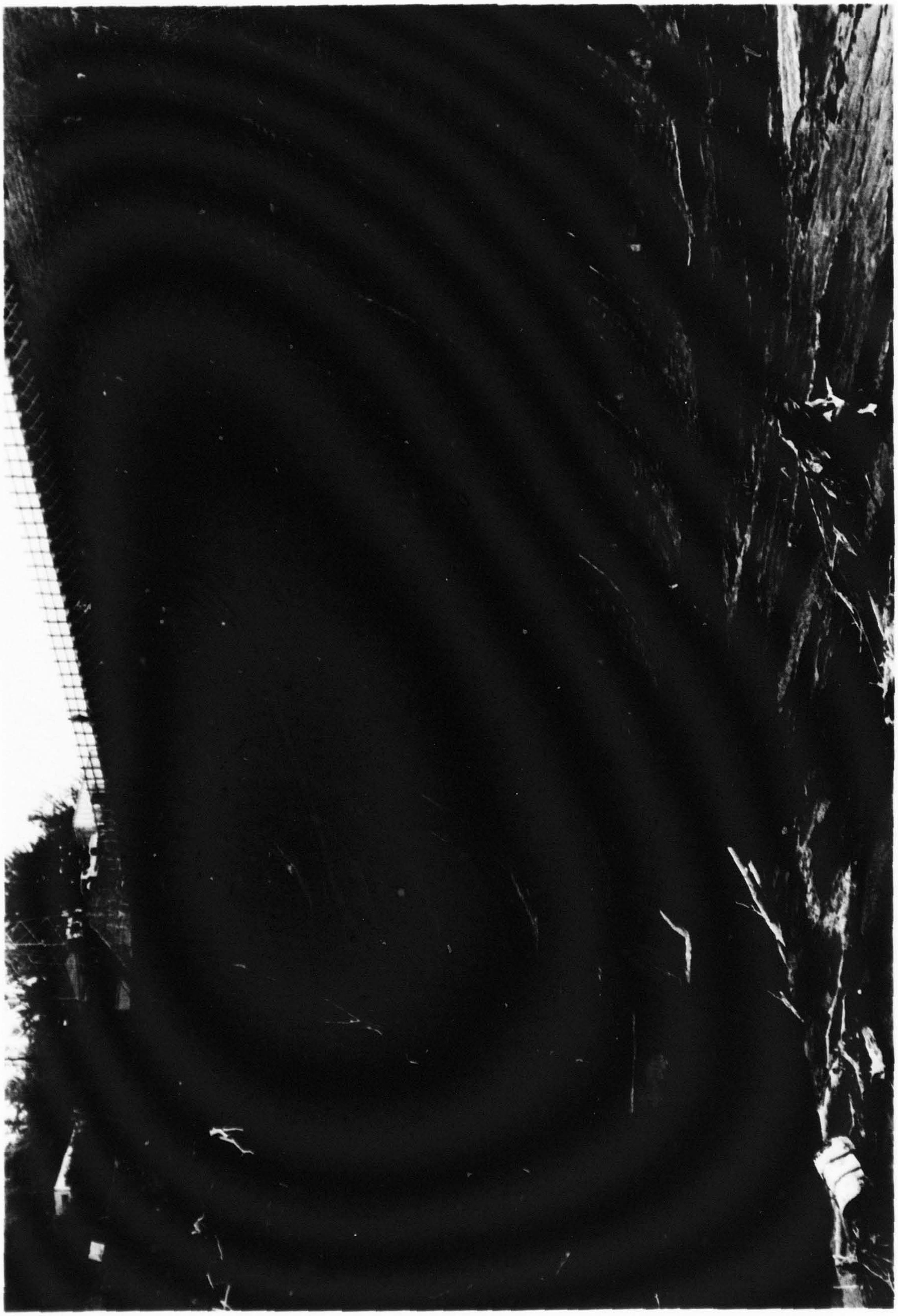


PLATE 1

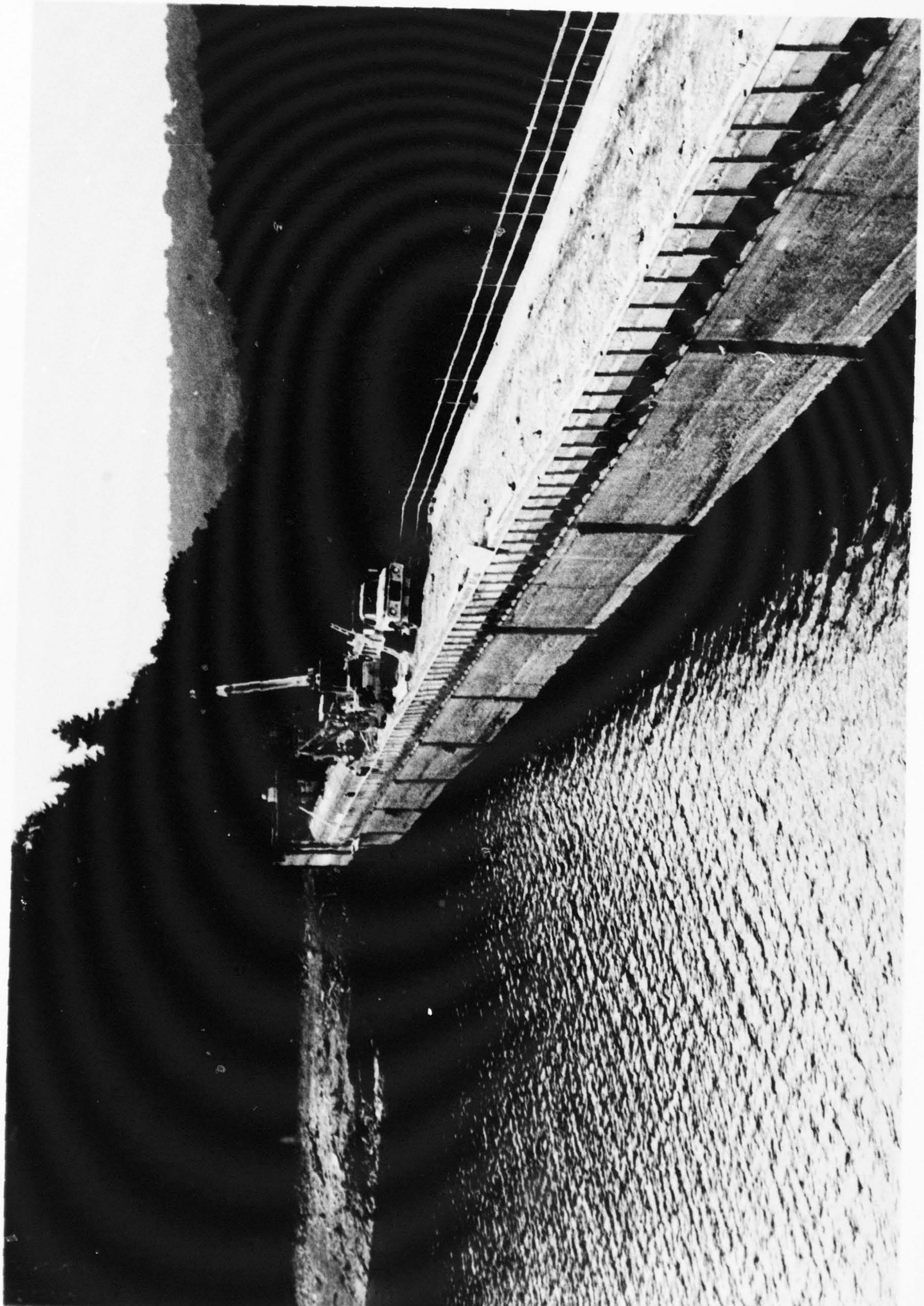


PLATE 2

PLATE 3

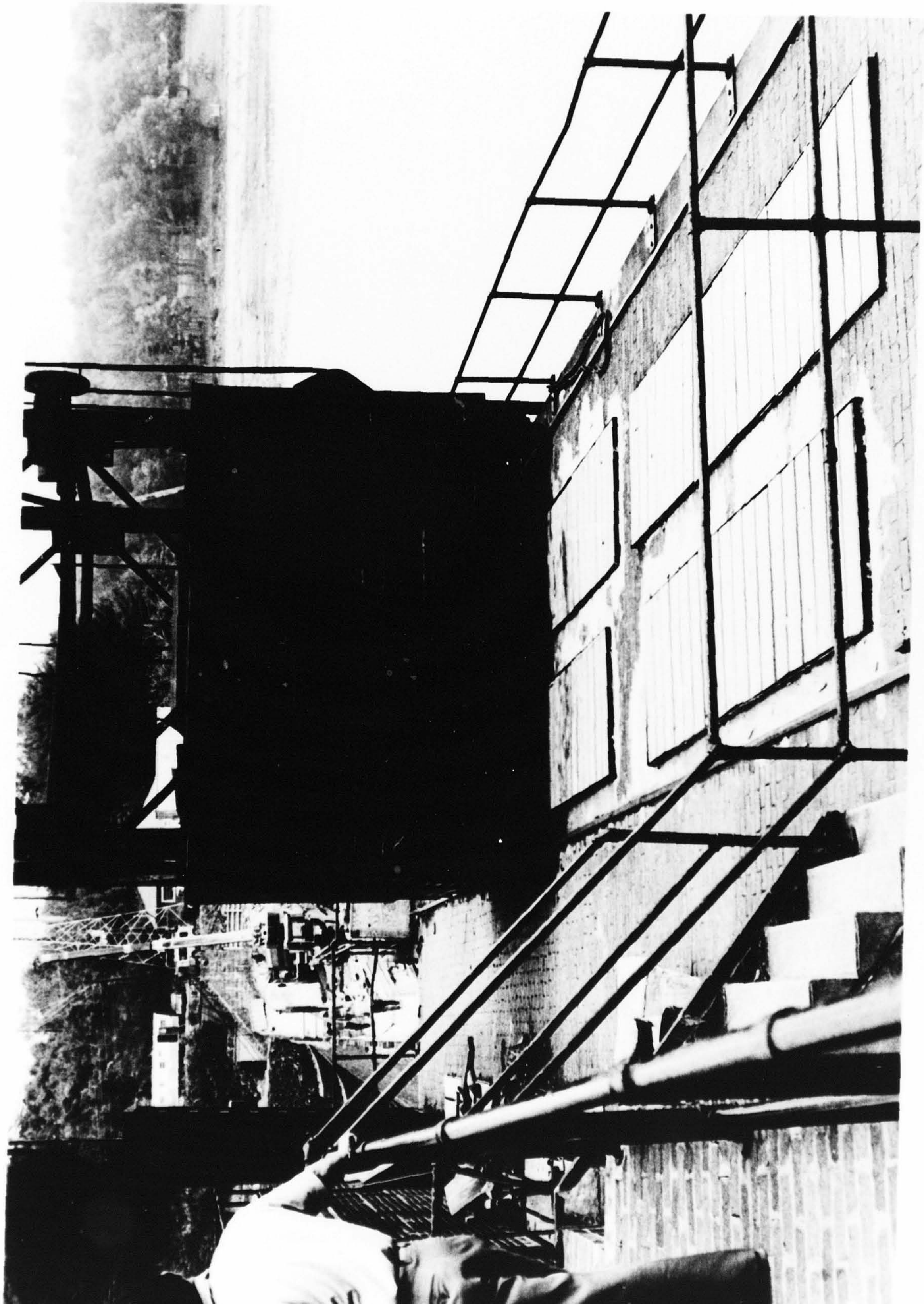


PLATE 4

PLATE 5

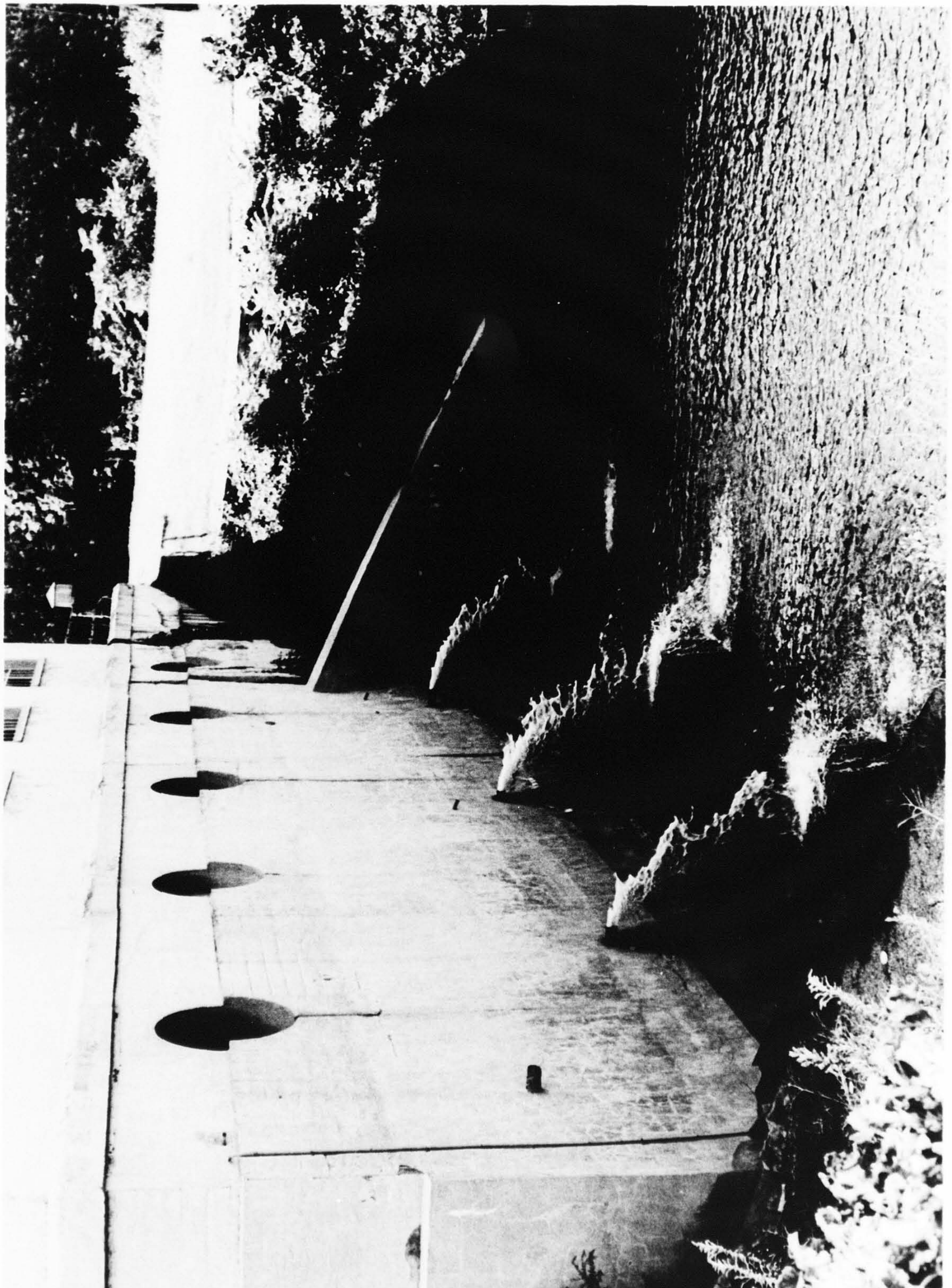


PLATE 6

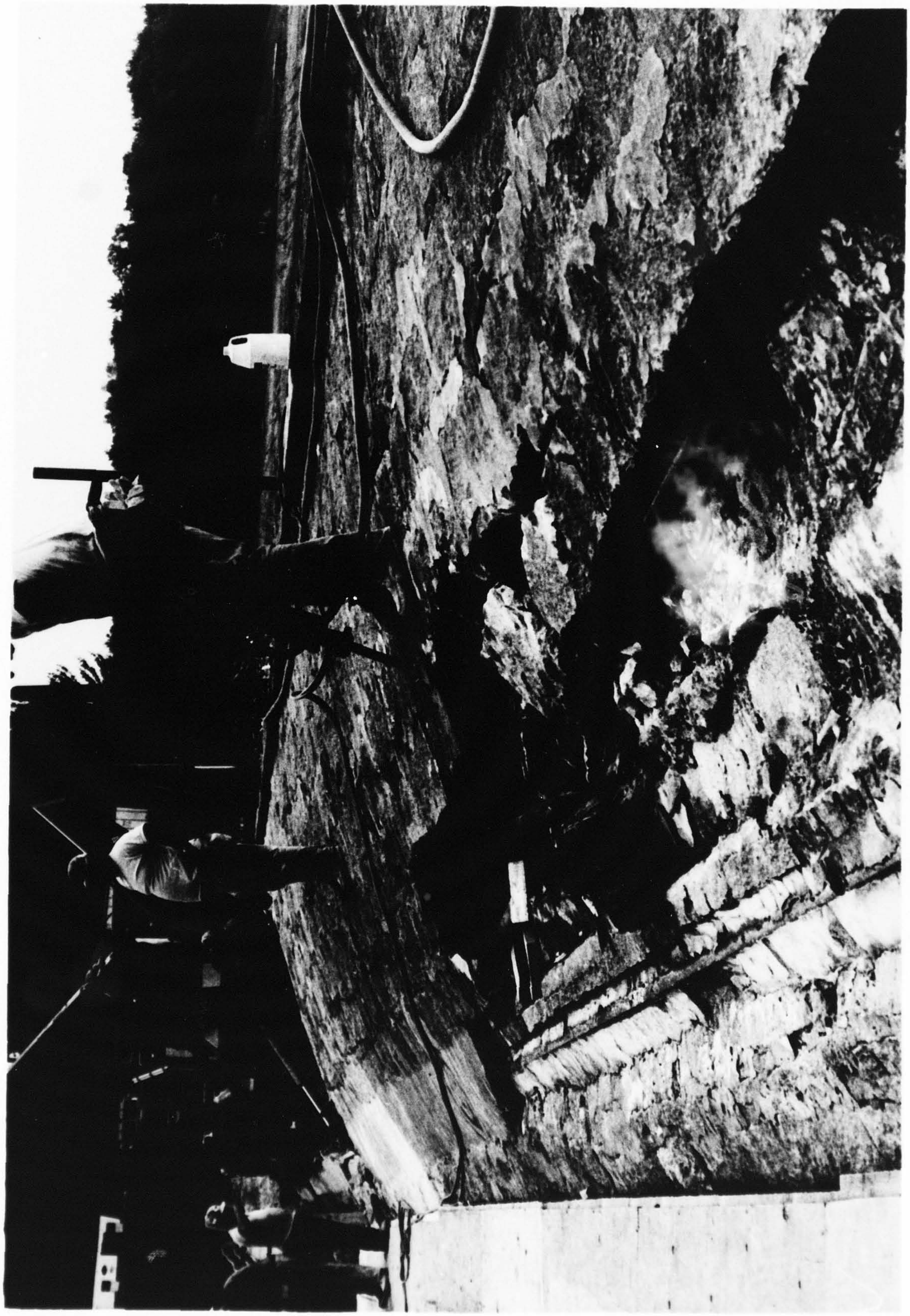


PLATE 7

PLATE 8

PLATE 9

APPENDIX D

PERTINENT CORRESPONDENCE AND REPORTS

PERMIT

UNDER THE ENVIRONMENTAL CONSERVATION LAW

[X] ARTICLE 15, (Protection of Water) [ ] ARTICLE 25, (Tidal Wetlands)
[ ] ARTICLE 24, (Freshwater Wetlands) [ ] ARTICLE 36, (Construction in Flood Hazard Areas)

T ISSUED TO Central Hudson Gas & Electric Corp. - Attn: Donald Otis			
ES OF PERMITTEE 284 South Avenue, Poughkeepsie, NY			
TION OF PROJECT (Section of stream, tidal wetland, dam, building) Sturgeon Pool			
IPTION OF PROJECT Reconstruction of concrete spillway in accordance with plans on file with this Department.			
U TY NAME (City, Town, Village)		TOWN Rosendale	
T Ulster	FIA COMMUNITY NO.	DAM NO. 778	PERMIT EXPIRATION DATE December 31,1979

GENERAL CONDITIONS

T permittee shall file in the office of the appropriate Regional ministrator, a notice of intention to commence work at least 48 in advance of the time of commencement and shall also notify him y in writing of the completion of the work.

T permitted work shall be subject to inspection by an authorized n ive of the Department of Environmental Conservation who may he work suspended if the public interest so requires.

As a condition of the issuance of this permit, the applicant has ac- e ressly, by the execution of the application, the full legal respon- all damages, direct or indirect, of whatever nature, and by whom- uffered, arising out of the project described herein and has agreed to ify and save harmless the State from suits, actions, damages and of ery name and description resulting from the said project.

/ material dredged in the prosecution of the work herein permitted e removed evenly, without leaving large refuse piles, ridges across the the waterway or flood plain or deep holes that may have a tendency to in y to navigable channels or to the banks of the waterway.

Any material to be deposited or dumped under this permit, either in terway or on shore above high-water mark, shall be deposited or dumped locality shown on the drawing hereto attached, and, if so prescribed ithin or behind a good and substantial bulkhead or bulkheads, such vent escape of the material into the waterway.

There shall be no unreasonable interference with navigation by the e n authorized.

t if future operations by the State of New York require an alteration position of the structure or work herein authorized, or if, in the opinion Department of Environmental Conservation it shall cause unreasonable t to the free navigation of said waters or flood flows or endanger l safety or welfare of the people of the State, or loss or destruction natural resources of the State, the owner may be ordered by the Depart- o remove or alter the structural work, obstructions, or hazards caused hout expense to the State; and if, upon the expiration or revocation rmit, the structure, fill, excavation, or other modification of the ourse hereby authorized shall not be completed, the owners shall, t expense to the State, and to such extent and in such time and manner Department of Environmental Conservation may require, remove all or n of the uncompleted structure or fill and restore to its former o he navigable and flood capacity of the watercourse. No claim shall de against the State of New York on account of any such removal or ion.

8. That the State of New York shall in no case be liable for any damage or injury to the structure or work herein authorized which may be caused by or result from future operations undertaken by the State for the conservation or improvement of navigation, or for other purposes, and no claim or right to compensation shall accrue from any such damage.

9. That if the display of lights and signals on any work hereby authorized is not otherwise provided for by law, such lights and signals as may be pre- scribed by the United States Coast Guard shall be installed and maintained by and at the expense of the owner.

10. All work carried out under this permit shall be performed in accor- dance with established engineering practice and in a workmanlike manner.

11. If granted under Articles 24 or 25, the Department reserves the right to reconsider this approval at any time and after due notice and hearing to continue, rescind or modify this permit in such a manner as may be found to be just and equitable. If upon the expiration or revocation of this permit, the modification of the wetland hereby authorized has not been completed, the applicant shall, without expense to the State, and to such extent and in such time and manner as the Department of Environmental Conservation may require, remove all or any portion of the uncompleted structure or fill and restore the site to its former condition. No claim shall be made against the State of New York on account of any such removal or alteration.

12. This permit shall not be construed as conveying to the applicant any right to trespass upon the lands or interfere with the riparian rights of others to perform the permitted work or as authorizing the impairment of any rights, title or interest in real or personal property held or vested in a person not a party to the permit.

13. The permittee is responsible for obtaining any other permits, ap- provals, lands, easements and rights-of-way which may be required for this project.

14. If granted under Article 36, this permit is granted solely on the basis of the requirements of Article 36 of the Environmental Conservation Law and Part 500 of 6 NYCRR (Construction in Flood Plain Areas having Special Flood Hazards — Building Permits) and in no way signifies that the project will be free from flooding.

15. By acceptance of this permit the permittee agrees that the permit is contingent upon strict compliance with the special conditions on the reverse side.

(SEE REVERSE SIDE)

Permittee is authorized to install a temporary construction ramp of gravel fill approximately 20' x 40' in dimensions. The ramp must be removed upon completion of the project.

7. Upon completion of construction, the applicant shall notify the Chief Permit Administrator in writing and shall submit a notarized statement from the design engineer to the effect that the project has been completely constructed under his care and supervision in accordance with plans and specifications.

UE DATE	PERMIT ADMINISTRATOR	ADDRESS
14, 1978	Stanford J. Zeccolo	Dept. of Environ. Conservation 50 Wolf Road, Albany, NY 12233

G. Danskin, G. Koch, Bureau of Soils, file

778 LH

ARMcK-F.

May 8, 1922.

Re Proposed Dam,
Lower Hudson.

The J. G. White Engineering Corporation,
43 Exchange Place,
New York City.

ATTENTION OF WM. P. CREAGER.

Gentlemen:

We have received your letter of May 4, 1922, requesting applications for the construction of a dam at Sturgeon Pool, which application blanks we enclose. The application should be filled out completely and submitted, together with plans in duplicate, for approval of this department before construction is commenced.

Very truly yours,

FRANK M. WILLIAMS,
State Engineer

By ____________
Assistant Deputy.

Enclosure

ARMcK-F.

July 18, 1922.

Dam No. 778,
L. Hudson

Hon. Alexander Macdonald,
Conservation Commissioner,
Albany, N. Y.

Dear Sir:-

The United Hudson Electric Corporation of Poughkeepsie wishes to construct a dam on Wall Kill one mile below Rifton. See U. S. G. S. Map No. 193. The dam will be of concrete 100 feet high by 670 ft. long and will be used for power purposes. Will this dam require a fishway?

Very truly yours,

FRANK M. WILLIAMS,
State Engineer

By____________________
Assistant Deputy.

ALEXANDER MACDONALD
COMMISSIONER

TRACY STAGG
DEPUTY COMMISSIONER

ROBERT F. PRESCOTT
SECRETARY

STATE OF NEW YORK

CONSERVATION COMMISSION

ALBANY

DIVISION OF FISH AND GAME
LLEWELLYN LEGGE, CHIEF
DIVISION OF LAND AND FORESTS
C. R. PETTIS, SUPERINTENDENT
DIVISION OF SARATOGA SPRINGS
J. G. JONES, SUPERINTENDENT
SARATOGA SPRINGS, N. Y.

IN REPLYING PLEASE REFER TO FILE NO.

McKim

July 19, 1922.

JUL 20 1922

Mr. Frank M. Williams,
State Engineer and Surveyor,
Albany, N. Y.

Attention Charles R. Waters, Asst. Deputy.

Dam #778. L. Hudson.

Dear Sir:

In answer to the above relative to dam mentioned it is respectfully requested that plans as stated in your letter of July 6th, omit the provisions for a fishway.

Very truly yours,

ALEXANDER MACDONALD, Commissioner,

By- Llewellyn Legge Per. K.C.

Chief, D.F.G.C.C.

LL/C

UNITED HUDSON ELECTRIC CORPORATION

50 MARKET STREET

POUGHKEEPSIE, NEW YORK

Field Office Rifton, N. Y.

July 18 1922.

ngineer,

. N. Y..

ir:

eferring to application for the construction of a dam on the Walkill River at Rifton

rk handed your Mr McKinn yesterday.

e are sending you herewith an additional blue print of J. G. White Engineering Corp.

: E 5276 - 4 E 5275 - 3 and E5310 - I.

Yours very truly,

United Hudson Electric Corporation,

F. B. Maltby

F. B. Maltby,
Resident Engineer.

r.

V

r Separate Cover.

ARMcK-F.

July 19, 1922.

Dam No. 778, L. Hudson

The United Hudson Electric Corporation,
50 Market Street,
Poughkeepsie, N. Y.

Gentlemen:

We have received from your engineer, F. B. Maltby, three prints in duplicate, marked H-5275-3, H-5276-4 and H-5310-1, and application for the construction of a concrete dam 100' high by 670' long at Sturgeon Pool on the Wallkill river, one mile below Rifton, this dam to be founded entirely upon sandstone ledge. We have marked this dam upon our records as No. 778, Lower Hudson Watershed.

We will require a report from your engineer on each section of the bed and banks as soon as excavated, concerning the character of the material, the hardness and perviousness; the roughness and shoulders to resist shear; the proposed dimensions of the cutoff walls and provisions against sliding and under seepage.

The construction of this dam is approved in so far as the matter involves the jurisdiction conferred upon this office by Chapter LXV of the Consolidated Laws, and Chapter 647, Laws of 1911, Section 22, and permission is given for the construction of this work up to November 2, 1924. This approval shall not be deemed to authorize any invasion of property rights, either public or private, in carrying out the above work; nor to create any claim against the State of New York; nor to be considered as authorizing the flooding of State lands, nor as acquiescing in the flooding of such lands.

If flashboards are to be used in the spillway, they should be so designed as to give way entirely before the pond level reaches two-thirds the depth of the spill, so that the whole spillway may be available for floods. The design of these flashboards, giving the span of the supports and dimensions of the parts, should be submitted to and approved by this office before they are used.

The U.N.E.Corp. #2 7/19/22

We return herewith one set of prints of plans submitted, stamped on the back with our approval.

Please acknowledge the receipt of this letter and advise us when you start the above work.

Very truly yours,

FRANK M. WILLIAMS,
State Engineer

By ______________________
Deputy State Engineer.

Copy to
Mr. F.B.Maltby,
c/o J.G.White Engr.Corp.
Rifton, N. Y.

Enclosure.

Shipping Tag & 2 given to Mr Maltby June 17. 1922.
G.K.H.

UNITED HUDSON ELECTRIC CORPORATION

50 MARKET STREET

POUGHKEEPSIE, NEW YORK

Field Office Rifton, N. Y.
July 21 1922.

RECEIVED JUL 22 1922

Fr[illegible]k M. Williams,

[illegible]te Engineer,

[illegible]an[illegible], N. Y.

[illegible] Sir:

I beg to acknowledge receipt of your favor of the 19th enclosing a copy of your

[illegible] of the plans for a dam on the Walkill River near Rifton, N. Y.

The distance from the junction of the Walkill and Rondout is 3100 feet measured

[illegible] the Walkill River.

Yours very truly,

United Hudson Electric Corporation,

F. B. Maltby

F. B. Maltby,
Resident Engineer.

[illegible]vf.

UNITED HUDSON ELECTRIC CORPORATION

50 MARKET STREET

POUGHKEEPSIE, NEW YORK

Field Office Rifton, N. Y.
July 25 1922.

Dam 778 LH

RECEIVED
OFFICE STATE ENG.
JUL 26 1922

Frank M. Williams State Engineer,

r , N. Y.

Sir:

We beg to acknowledge the receipt of your favor of the 19th inst approving the con-

struction of a dam on the Walkill River at Sturgeon Pool one mile below Rifton, New York.

One set of prints H 5310 , H 5276 - 4 and H 5275 - 3 which have been stamped as

ed by you are also received.

Your requirements as to a report by the Resident Engineer as mentioned in Paragraph

your letter are noted and will be complied with.

A contract for the construction of the dam has been entered into with the Foundation

pany of New York and they are now moving in their plant.

Yours very truly,

United Hudson Electric Corporation,

F. B. Maltby

F. B. Maltby,
Resident Engineer.

f.

State of New York
Department of State Engineer and Surveyor
Testing Laboratory
Albany

Tests of Sand from ~~bank at~~ Rifton N. Y.,
For use on ~~Contract No.~~ Dam of United Hudson Electric Corp. ~~Canal~~ at Rifton Division.
Contract Sample No. — taken July 21; received at Laboratory July 26; made up July 28
Sand is composed mainly of fine grains of quartz and feldspar with some limestone and granite grains – slightly coated with loam.
Percentage of Voids 35.8; Loam 3.3; Organic matter ——
Parts of sand to cement by ~~weight~~ bulk :— 2½ sand to 1 cement. Per cent water used 11
Temperature of water used in mixing 73 Fahr. Briquettes kept in moist air 24 hours and then immersed.
Cement used in tests, Standard Blend This cement tested as follows:—
Sets (determined by Vicat needle):—Initial, {in 85 min. / Minim. requirement 45 min.} ; hard, {in 410 min. / Max. requirement 600 min.}
Constancy of Volume Tests:—Normal air Good; Normal water Good; Accelerated Good
Fineness (per cent passing standard sieve No. 100) 98.0 (Requirement, 92%)
" (" " " " " No. 200) 83.7 (Requirement, 78%)

TENSILE STRENGTH IN POUNDS PER SQUARE INCH						SIZE OF SAND	
STANDARD SAND		NATURAL SAND		WASHED SAND		PASSING SIEVE	
7 Days	28 Days	7 Days	28 Days	7 Days	28 Days	No.	Per Cent
232	350	198		149		2(½")	100.0
258	361	208		152		4(¼")	99.0
231	340	216		156		6	98.6
202	343	196		165		10	96.6
248	341	192		164		20	85.0
1171	1735	1010		786		30	61.0
234	347	202		157		40	24.6
						60	4.6
						74	2.7
						100	1.5
						140	1.0
						200	0.6

Remarks: This sand is of uniform size grains.

I Certify *that this is a true abstract taken from the records of tests* August 7 1922

Russell S. Greenman
Sr. Ass't Engineer in Charge of Tests

ARMcK-M

August 8, 1922.

Subject - Dam 778
Lower Hudson-Rifton

United Hudson Electric Corporation
Rifton,
New York.

Gentlemen:

Concerning the sample of sand for the construction of dam 778 Lower Hudson at Rifton, the following is the result:

The sand is composed mainly of fine grains of quartz and felspar with some limestone and granite grains slightly coated with loam. This sand is of uniform size grains. The percentage of voids is 35.8 and of loam 3.3. The average of 5 - 7 day tests of 2½ sand to 1 of cement gave for the standard sand 234 pounds, for the natural sand as submitted by you 202 pounds and for the sand submitted by you and washed 157 pounds per square inch.

Yours very truly,

FRANK M. WILLIAMS,
State Engineer,

BY________________
Assistant Deputy.

UNITED HUDSON ELECTRIC CORPORATION

50 MARKET STREET

POUGHKEEPSIE, NEW YORK
P. O. Box 178 Kingston, N. Y.
March 19 1923.

St te Engineer,
Al any, New York.

OFFICE
MAR 20 1923
REF'D TO
ANS'D

Si :

Referring to your letter of July 19 1922 Dam No 778 L Hudson approving the construction of a Dam across the Wallkill River one mile below Rifton, N. Y.

In accordance with the second paragraph of your letter I am sending you a tracing showing the amount of excavation etc at sections 2 + 70 , 3 + 20 , 3 + 70 , 4 + 20 , 4 + 70 n at 5 + 20. The plan of the Dam on the same sheet shows the location of theses sections.

The excavation has been carried down in each instance to a hard compact very fine grain-d sandstone and entirely impervious.

The excessive excavation at station 2 + 70 was necessary on account of the broken charc er of the rock.

Practically all of the excavation is in rock except the left hand end of section 4 + 70 nd the right hand end of section 5 + 20, which are in an old canal bank. Section 5 + 70 is h floor of an old quarry. Sections not shown are not yet completed.

Yours very truly,

United Hudson Electric Corporation,

F. B. Maltby
F. B. Maltby,
Resident Engineer.

bm/cvf.

ARMcK-P.

March 22, 1923.

Dam No. 778, L. Hudson,
Rifton.

United Hudson Electric Corporation,
P. O. Box 178,
Kingston, N. Y.

ATTENTION OF F. B. MALTBY, RESIDENT ENGINEER.

Gentlemen:

We have received your letter of March 19th, and a tracing of the cross section of the bed excavations, dam No. 778, L. Hudson at Rifton.

Where the ledge bed is without shoulders capable of resisting shear from the dam section, we recommend that the dam section be carried down near the downstream edge into a hard, homogeneous and unlaminated rock bed for at least 1/20 of the vertical distance from the proposed upstream highest water surface to the natural bed, and that the upstream cutoff be carried down twice this distance.

Very truly yours,

....
Deputy State Engineer.

UNITED HUDSON ELECTRIC CORPORATION

50 MARKET STREET

POUGHKEEPSIE, NEW YORK

P. O. Box 178 Kingston, N. Y.
March 26 1923.

...eer,
...ew York.

... acknowledge the receipt of your favor of the 22nd concerning Dam No 773 L

... appreciate very much your suggestions.

... forming the base of the Dam is more irregular than my sections show as there

...houlders from 1 to 3 feet high.

...mendations will be followed as closely as possible.

Yours very truly,

United Hudson Electric Corporation,

F. B. Maltby

F. B. Maltby,
Resident Engineer.

The above information is correct to the best of my knowledge and belief.

Rifton New York
(Address of signer)

July 17-22
(Date)

F. B. Whatley — Malt by

Resident Engineer
(A person signing for Applicant should indicate his title or authority).

UNITED HUDSON ELECTRIC CORPORATION

50 MARKET STREET

POUGHKEEPSIE, NEW YORK

P. O. Box 178 Kingston, N. Y.

March 19 1923.

MAR 2[illegible] 1923

REF'D TO..................

ANS'D..................

ita e Engineer,
.lbany, New York.

ir

Referring to your letter of July 19 1922 Dam No 778 L Hudson approving the construction f Dam across the Wallkill River one mile below Rifton, N. Y.

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Practically all of the excavation is in rock except the left hand end of section 4 + 70 d the right hand end of section 5 + 20,which are in an old canal bank. Section 5 + 70 is e floor of an old quarry. Sections not shown are not yet completed.

Yours very truly,

United Hudson Electric Corporation,

F. B. Maltby

F. B. Maltby,
Resident Engineer.

m/cvf.

CENTRAL HUDSON GAS
AND
ELECTRIC CORPORATION
STURGEON POOL HYDRO GENERATING PLANT

REPORT ON
PROPOSED PLANT
RETIREMENT CONSIDERATIONS

MAIN
CHAS. T. MAIN OF NEW YORK, INC.

December 1973

1050-31

MAIN
Engineers

CHAS. T. MAIN OF NEW YORK, INC.
LINCOLN BUILDING, 60 EAST 42ND STREET, NEW YORK, NEW YORK 10017

PLEASE REPLY TO
Southeast Tower
Prudential Center
Boston, Mass. 02199
Telephone 617-262-3200

December 14, 1973 1050-31

SUBJECT: Sturgeon Pool Hydro Generating Plant
Report on Proposed Plant Retirement Considerations

Mr. Benon Budziak
Production Operations Superintendent
Central Hudson Gas & Electric Corporation
284 South Avenue
Poughkeepsie, New York 12602

Dear Mr. Budziak:

In accordance with your request we are pleased to submit herewith our report on the subject hydroelectric plant retirement considerations.

The purpose of this report is that of delineating and recommending the treatment of all major items judged to be required in the orderly retirement of the project structures and property in general, and to provide safe and stable structures with minimal maintenance requirements after retirement.

The scope of the work encompassed in the studies of this project included: a flood study of the Wallkill River to determine the Probable Maximum Flood (PMF) and a Standard Project Flood (SPF) with associated headwater and tailwater elevations, a structural stability analysis of the water retaining structures for the proposed normal operating condition and flood (SPF) condition with alterations considered for the retirement of this plant, field inspections and a survey to obtain necessary data for alterations, the preparations of plans and specifications detailing the work required to alter the structures, and an engineering cost estimate of the alteration work.

From the stability analyses of the water retaining structures, results indicate that all of the structures are stable under the loadings assumed. The structural analyses were based on currently accepted analytical practice and assumptions.

The total estimated cost of the construction work recommended for altering the project structures is $321,000. If it is decided to omit the powerhouse from the retirement alterations the total cost would be $213,600. These costs represent current present-day costs with no allowance for escalation.

We trust that you will find this report satisfactory and complete. Should any of the report content require further discussion or clarification, please do not hesitate to contact us.

Respectfully submitted,

CHAS. T. MAIN OF NEW YORK, INC.

J. C. Matte

J. C. Matte

JCM/es

AD-A069 101 KIMBALL (L ROBERT) AND ASSOCIATES EBENSBURG PA F/G 13/2
NATIONAL DAM SAFETY PROGRAM. STURGEON POOL DAM (INVENTORY NUMBE--ETC(U)
SEP 78 R J KIMBALL DACW51-78-C-0025

UNCLASSIFIED NL

2 OF 3

AD
A069101

TABLE OF CONTENTS

TABLES

APPENDIX

INTRODUCTION

The following report summarizes the findings in regard to alterations recommended for the retirement of the Sturgeon Pool Hydro Generating Plant. The report also summarizes the results of a flood analysis and a structural stability analysis of the water retaining structures, and includes an engineering cost estimate of the proposed alteration work.

FIELD INSPECTION & FIELD SURVEY

The existing structures and surrounding area of the Sturgeon Pool Hydro Generating Plant were inspected to determine the extent of alterations required to retire the plant and to obtain measurements to supplement the existing drawings for the preparation of the alteration drawings. The river and its bank, both upstream and downstream of the project were examined to see if any natural or man-made obstructions were present that might affect and impede the river flow, affecting headwater and tailwater levels. Photographs were taken to assist and supplement the obtained information.

In general, it was found that certain modifications to the existing structures need to be performed relative to plant retirement. The following modifications are recommended:

1. Remove the powerhouse superstructure and all mechanical and electrical equipment including the generators and place concrete slabs over the openings.

2. Remove the three steel penstocks including all concrete supports, regrade the area, and plant grass seed.

3. Plug the water passageways of the intake structure with concrete and fill the openings left by the penstocks' removal with concrete block masonry.

4. Place reinforced concrete slabs over openings in the intake structure.

5. Plug the six water sluiceways in the ogee spillway dam with concrete.

6. Place chain link fencing on the left non-overflow dam structure.

7. Place rockfill in the draft tube openings.

8. Install handrailing along the walkway to the upper inspection gallery entrance.

9. Remove the bulkhead section and modify the spillway crest in the area of the existing upper level sluice gate and install a new sluice gate.

10. Install splashboards along the existing spillway training wall.

11. Remove the upstream crane rail on the ogee spillway and restore the area with concrete.

HYDROLOGY & FLOOD STUDY

The Sturgeon Pool Hydro Generating Plant is located on the Wallkill River immediately downstream from the Dashville Hydro Generating Plant and commands about 20 square miles of additional drainage area, or 805 square miles. Because of this close proximity, it was assumed that the Dashville Standard Project Flood (SPF) routed outflow hydrograph would be the inflow hydrograph to Sturgeon Pool since the storm runoff from the small intervening drainage area would have probably peaked well before the basin SPF entered the reservoir. The peak flow of 38,000 cfs was undiminished and it transited the project pond with a maximum elevation of 137.3 and a maximum tailwater elevation of 42.0.

Further information on the Hydrology and Flood Study of Dashville and Sturgeon Pool may be obtained from the Dashville Report, March, 1973.

STRUCTURAL STABILITY ANALYSIS

A structural stability analysis of all water retaining structures in their final altered condition was performed to verify that the structures would be stable under the assumed loadings as presented in Table I. The results of the analysis, as presented in Table III, indicate that the structures are stable under the loadings as assumed. Tables I and II list the loading cases checked and values and assumptions used in the above analysis.

The safety factors of the structures were checked with respect to overturning and sliding at their bases, and computations were made to check the foundation pressures at this elevation.

The safety factor with respect to overturning is the ratio of the forces (the weight of structure) times their lever arms (moments) tending to prevent the structure from tipping to the forces (moments) tending to tip the structure (the water pressure exerted on the upstream face and beneath the structure). Any safety factor that is equal to 1.0 would theoretically indicate that the structure is stable, with any lesser value placing it at the verge of being unstable. By refering to Table III, under the column headed Σ Mr/ Σ Mo, for all cases considered, the structures are stable with respect to overturning.

The safety factor with respect to sliding including shear-friction resistance is the ratio of the forces tending to resist sliding; namely, the frictional resistance due to the net weight of the structure sliding along its base and the resistance due to the shearing strength developed between the structure and its rock foundation, and the forces tending to promote sliding; namely, the water pressure at the upstream face. It is normally accepted that this ratio be as a minimum 5.0. By refering to Table III, under the column headed S_{s-f} for all cases considered, the structures are stable with respect to sliding.

TABLE I

CASES USED
STABILITY ANALYSIS

Case I	Normal Water Levels (proposed) H.W.L. = 129.5 T.W.L. = 20.0 Uplift Included
Case II	Standard Project Flood Water Levels H.W.L. = 137.3 T.W.L = 42.0 Uplift Included

TABLE II

VALUES AND ASSUMPTIONS
STABILITY ANALYSIS
CONCRETE SECTIONS

1. Nomenclature:

ΣH	=	Summation of Horizontal Forces
ΣV	=	Summation of Vertical Forces
ΣM_R	=	Summation of Resisting Moments
ΣM_o	=	Summation of Overturning Moments
$\frac{\Sigma M_R}{\Sigma M_o}$	=	Factor of Safety Against Overturning
$\frac{\Sigma H}{\Sigma V}$	=	Coefficient of Sliding

2. Unit weight of concrete - 150 lbs/cu. ft.

3. Unit weight of water - 62.4 lbs/cu. ft.

4. Uplift Pressure: The pressure was assumed to vary linearly from full headwater pressure at the upstream side to full tailwater pressure at the downstream side taken over 100% of the base area.

5. Sliding (Shear Included): For a discussion and explanation of terms, see *Hydroelectric Handbook* by Creager and Justin, John Wiley & Sons, Inc., Second Edition - Page 341.

$$S_{s\text{-}f} = \frac{f \, \Sigma V + r \, Sa \, A}{\Sigma H}$$

Where:

$S_{s\text{-}f}$	=	Shear Friction Factor of Safety
f	=	0.75
r	=	0.5
Sa	=	380 psi
A	=	Area of base

TABLE III

STABILITY SUMMARY

SECTION	CONDITION	BASE ELEV.	BASE LENGTH	ΣH (KIPS)	ΣV (KIPS)	$\frac{\Sigma H}{\Sigma V}$	S_{s-f}	RESULTANT FROM DOWNSTREAM	ΣM_R (K-FT)	ΣM_O (K-FT)	$\frac{\Sigma M_R}{\Sigma M_O}$	BASE STRESS (PSI) UPSTREAM	BASE STRESS (PSI) DOWNSTREAM
OGEE SPILLWAY	I - NORMAL	20.0	90.0	18,703	22,760	0.82	7.5	19.5	2,048,105	1,605,129	1.28	- 24.7	94.9
	II - FLOOD	20.0	90.0	20,589	19,987	1.03	6.7	36.3	2,081,120	1,906,652	1.09	- 43.8	105.4
INTAKE STRUCTURE	I - NORMAL	84.75	53.75	1,250	2,506	0.50	25.0	25.6	146,755	82,547	1.78	13.9	18.4
	II - FLOOD	84.75	53.75	1,723	2,128	0.81	17.7	14.9	146,755	115,150	1.27	- 4.7	32.2
NON-OVERFLOW DAM	I - NORMAL	104.5	26.86	19.5	58.5	0.33	39.9	15.4	1,442	539	2.68	16.7	8.3
	II - FLOOD	104.5	26.86	33.6	52.1	0.64	23.1	11.2	1,442	861	1.68	6.7	20.3

NOTE:

1. NEGATIVE BASE STRESS INDICATES TENSION

CENTRAL HUDSON GAS & ELECTRIC CORP.
POUGHKEEPSIE, NEW YORK

STURGEON POOL HYDRO

CONCRETE SECTIONS ANALYZED FOR STABILITY

MAIN

ALTERATIONS REQUIRED FOR RETIREMENT

Intake Structure

As part of the recommended alteration work required to retire the Sturgeon Pool Hydro Generating Plant, it is recommended that all trash screens and rakes, moving gate crane and penstocks be removed.

After the above items have been removed, it is further recommended that the following alterations are attended to: (1) construct reinforced concrete slabs over the intake gates and stop log recesses; (2) construct concrete plugs in the intake passageways to permanently seal them since the existing gates cannot be depended upon to serve indefinitely; (3) seal vent openings and openings left by the penstock removal with concrete block and mortar; and (4) clean and seal the manhole inspection covers on the intake deck. The final "altered" structure should be practically maintenance free and is believed to present no danger to the public. Plate II, included in the Appendix, depicts the above discussed alteration work.

Ogee Spillway Dam

As part of the retiring alteration work, it is also recommended that all electric motors and accompanying wiring and steel flooring be completely removed from the low level sluiceway valve chambers. After the above items have been removed it is recommended that concrete plugs be placed in the sluiceway and in the valve chambers. Plate III depicts the recommended method of doing this alteration work.

It is also recommended that the flashboards and pins be completely removed from the dam, however, it should by noted that the removal of these should be delayed until the work required to place concrete for the sluiceway and valve chamber plugs is completed. This would allow the Contractor to use the boards if he so chooses in his scheme of diverting water to provide access to the sluiceways and the passageway to the valve chambers. After the boards are removed, regulation of the pond will be determined by Nature and maintenance will be minimal.

In order to facilitate future maintenance work of the granite block protective facing covering the spillway face, modifications to the spillway dam in the area of the existing skimmer gate is required. By providing the alterations as depicted on Plate I, river discharges of approximately 400 cfs can be passed through this area which are exceeded on the average only 40 percent of the time during the months of July, August, and September, and at a level of approximately elevation 126.5; 2.0 feet $\pm$ below crest level.

The above alteration will allow most river flows that occur during the given months to be diverted away from the work area, thus limiting the amount of sand bagging required in the maintenance operation and providing for a relatively dry work area on the spillway face. The work would consist of removing the existing crest concrete to the elevations as shown to facilitate the placement of a new reinforced concrete cap and the installation of a new sluice gate. In addition to the above modifications, the splashboards in the area designated on the plans should be extended as detailed to prevent water from overtopping the structure when the sluice gate is operating.

As another part of the alteration work it is also recommended that the crane rail located upstream of the crest be removed and the area restored as indicated on the plans.

Penstocks

Further alteration work required would include the complete removal of the steel penstocks and their concrete supports from the intake structure to the face of the powerhouse. After these items have been removed, it is recommended that the area be rough graded, 6 inches of selected fill material placed over the area, and the area seeded. Plates I, II & III show the extent and details of the penstock removal.

Powerhouse

It is recommended that the entire superstructure of the powerhouse including the interior walls and furnishing be removed to generator floor level. In addition to this work, all mechanical and electrical apparatus above generator floor level should be removed. The latter would include removal of the following items: (1) the three generators and exciters; (2) switchgear and all electrical control panels; (3) oil, oil storage tanks, pumps and connecting piping; (4) the powerhouse crane, and (5) miscellaneous equipment.

After removal of the items listed above, it is further recommended that the following alterations are attended to: (1) place rockfill in the draft tube and tailrace; and (2) construct reinforced concrete slabs over the turbine pit openings, trenches, stair and manhole openings. The structure so modified should be virtually maintenance free and is believed to present no danger to the public. Plate IV included in the Appendix depicts the above discussed alteration work. If at a later date it is decided not to remove the powerhouse superstructure, Plate IV and the relevant items listed in the specifications could be removed or deleted. No studies have been made with regard to converting the powerhouse to other uses, however, a revised cost estimate for the omission of the powerhouse alterations has been included in this report.

Left Abutment Fencing and Walkway Handrail

It is recommended that fencing be fitted to the top of the non-overflow dam. This fencing consists of a chain link fence topped with 3-strands of barbed wire to provide further safety to the public and should deter all but the most persistent trespasser. The layout and details of the fence are depicted on Plate II in the Appendix.

It is also recommended that a new handrail be installed paralleling the walkway to the intake structure and inspection gallery for the spillway dam, to replace the existing railing that is attached to the penstock support saddles. The location and details of the proposed handrail system is shown on Plate I.

PLANS AND SPECIFICATIONS

Plans and specifications for the alteration work have been prepared to enable a Contractor to carry out the proposed work. Copies of the plans are bound with the specifications and the entire work entitled: Central Hudson Gas and Electric Corporation, Sturgeon Pool Hydro Generating Plant, Specification No. 1050-31A for Retiring Alterations to the Powerhouse and Associated Facilities in the Town of Rosendale, New York. The specifications were prepared in such a way that the award of the Contract by Central Hudson would be on a gross sum bid of eleven separate work items and would establish unit prices for two such items that could vary depending on field conditions. Quantities for these two items, used for cost estimating purposes, were based on the furnished construction drawings and the information obtained from the field inspection and survey trips.

ALTERATION COST ESTIMATE

A cost estimate of the alteration work is included in this report as Table IV and is believed to be a representative estimate of the construction costs that would be incurred during the year 1973. The items in the cost estimate follow the items included in the Specification Bid Schedule. The total estimated cost of the required alteration is $321,000.

If it is decided at a later date that the type and extent of the proposed alteration work to the powerhouse would not include the work as shown by Plate IV, the total estimated cost of the required alterations would be reduced by $107,400. A cost estimate delineating the remaining proposed alterating work is presented in Table V, amounting to a total cost of $213,600.

TABLE IV

COST SUMMARY
FOR
RETIRING ALTERATIONS
TO
THE STURGEON POOL HYDRO GENERATING PLANT

Item No.	Work or Material	Cost
1.	Concrete plugs in penstocks at intake and in sluiceways at spillway dam.	$ 85,000
2.	Concrete slabs on powerhouse generator deck and intake deck.	12,000
3.	Removal of powerhouse superstructure and equipment for powerhouse, and intake/dam moving gate crane.	79,000
4.	Rock fill for draft tubes.	6,000
5.	Removal of penstocks, foundations, and related work.	30,000
6.	Concrete block masonry in powerhouse substructure walls and at penstock and vent openings at intake structure.	2,000
7.	Chain link fencing.	1,000
8.	Handrailing.	2,600
9.	Removal of existing concrete, placing new concrete cap and installing new crest sluice gate.	35,000
10.	Install new splashboards.	1,000
11.	Removal of crane rail and related work.	3,300
	Subtotal	$256,900
	Contingencies (25%)	64,100
	Total Alteration Cost	$321,000

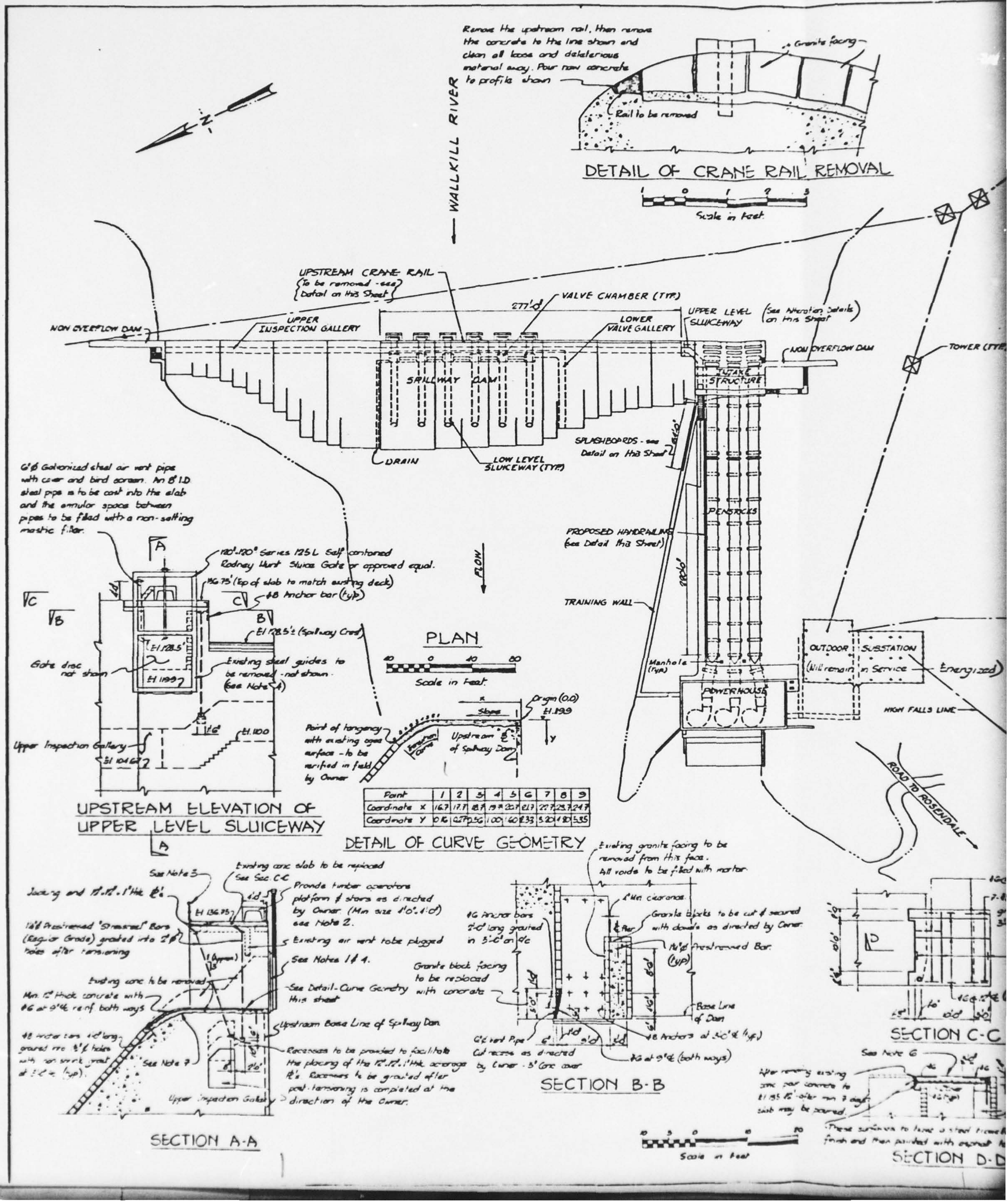
TABLE V

CONST SUMMARY
FOR
RETIRING ALTERATIONS
TO
THE STURGEON POOL HYDRO GENERATING PLANT

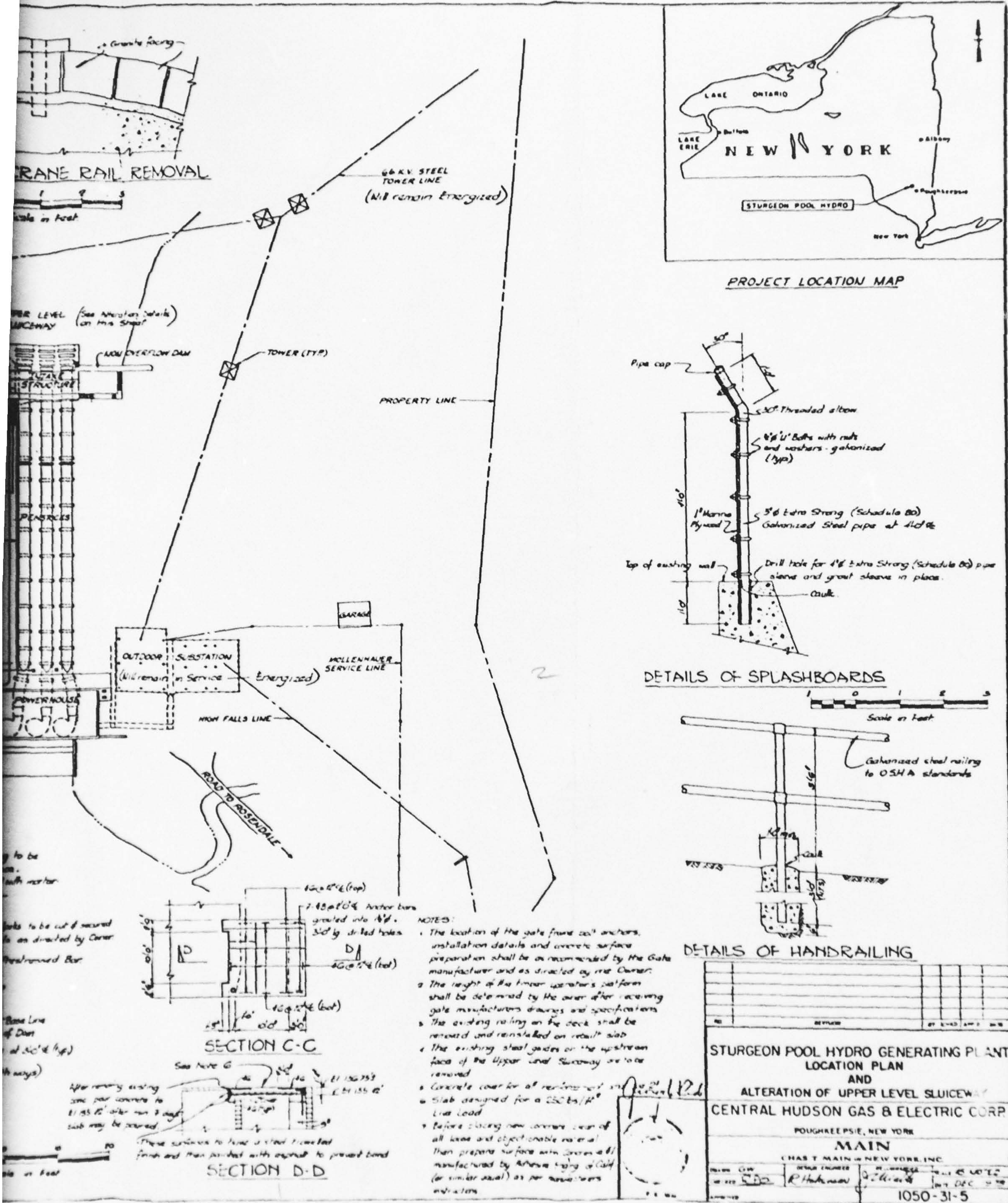
(Powerhouse Alterations Deleated)

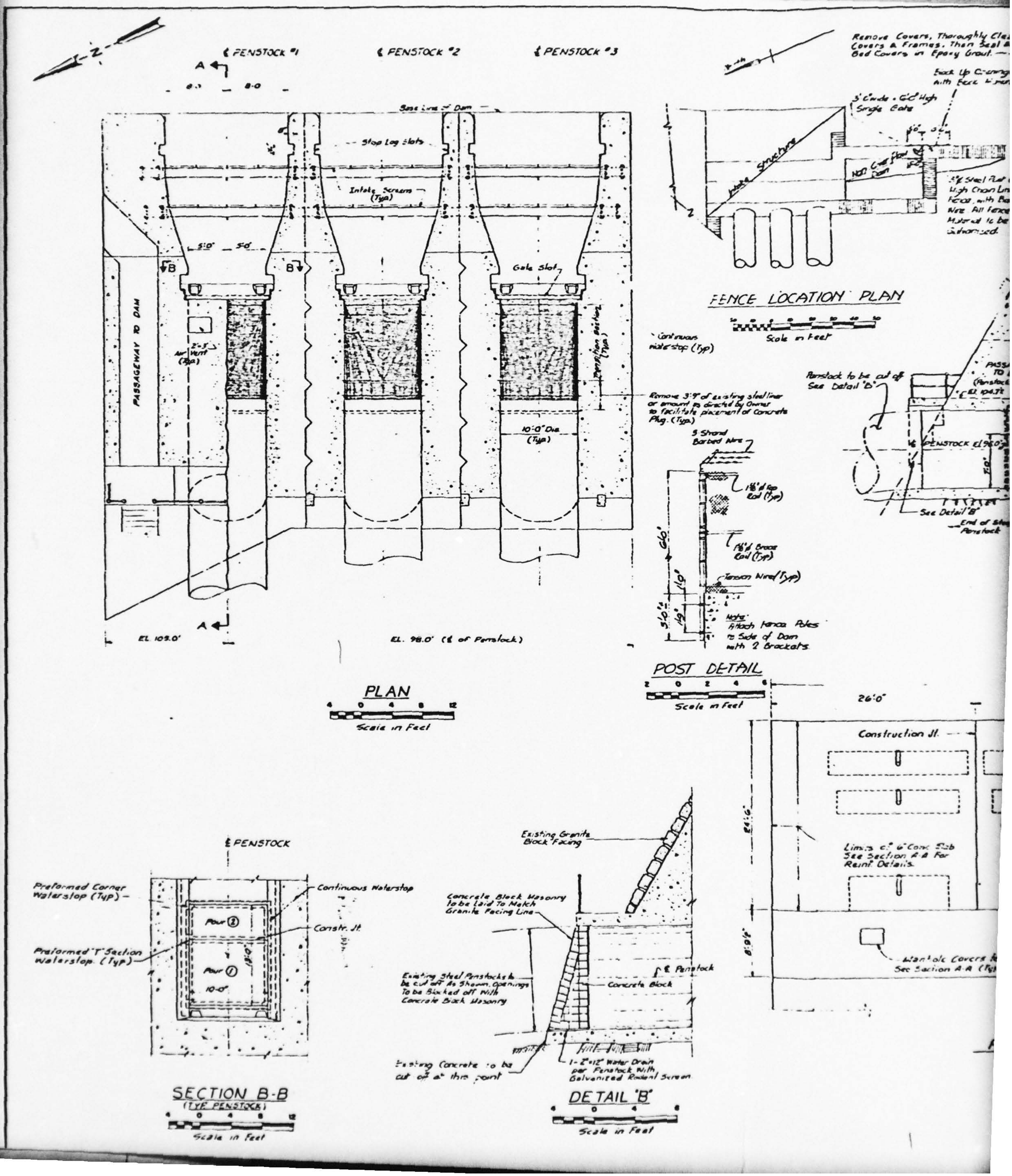
Item No.	Work or Material	Cost
1.	Concrete plugs in penstocks at intake and in sluiceways at spillway dam.	$ 85,000
2.	Concrete slabs on intake deck.	5,000
3.	Removal of equipment for intake/dam and moving gate crane.	6,000
4.	Removal of penstocks, foundations and related work.	30,000
5.	Concrete block masonry in penstock and vent openings at intake structure.	2,000
6.	Chain link fencing.	1,000
7.	Handrailing	2,600
8.	Removal of existing concrete, placing new concrete cap, and installing new crest sluice gate.	35,000
9.	Install new splashboards.	1,000
10.	Removal of crane rail and related work.	3,300
	Subtotal	$170,900
	Contingencies (25%)	42,700
	Total Alteration Cost	$213,600

APPENDIX

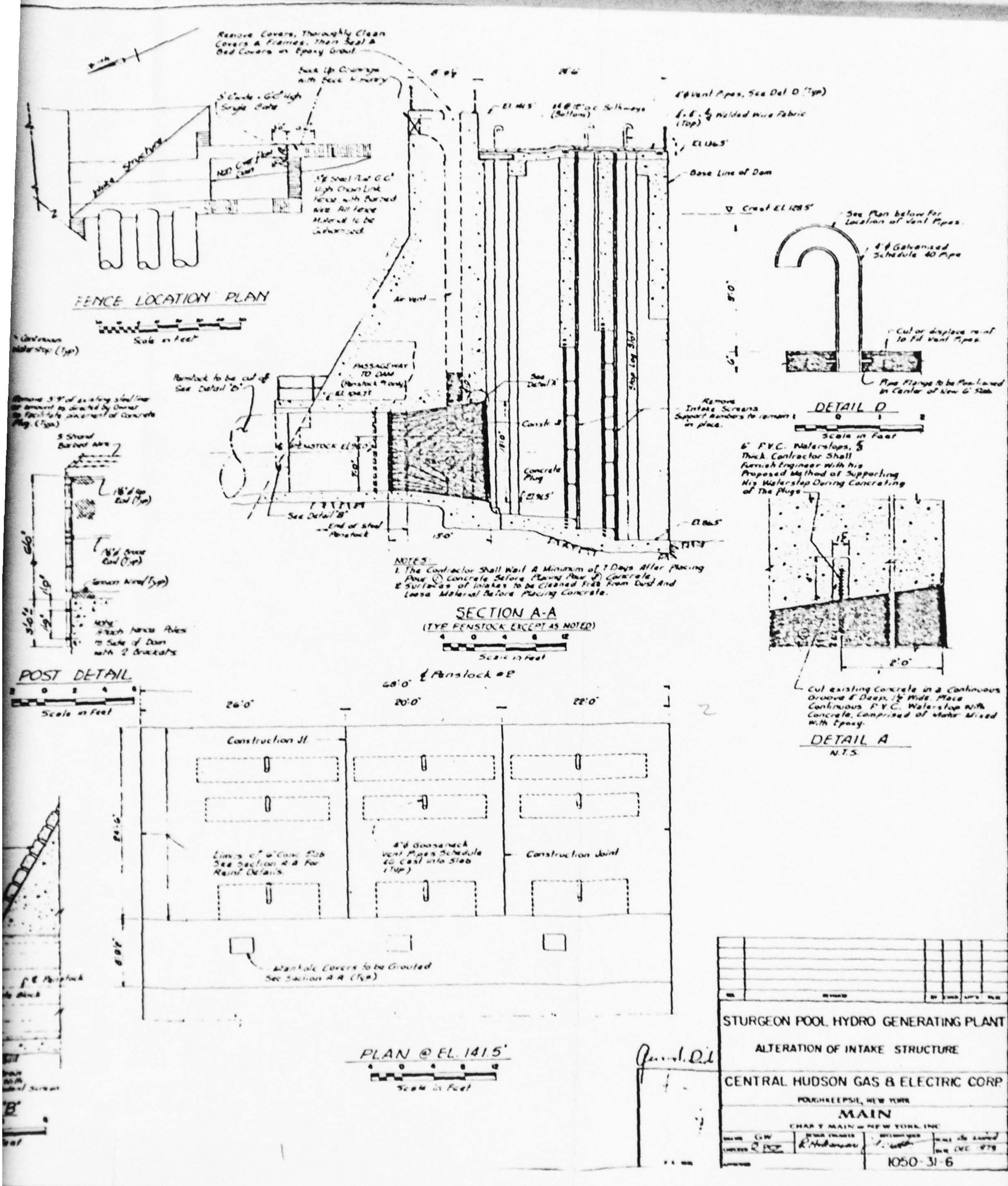


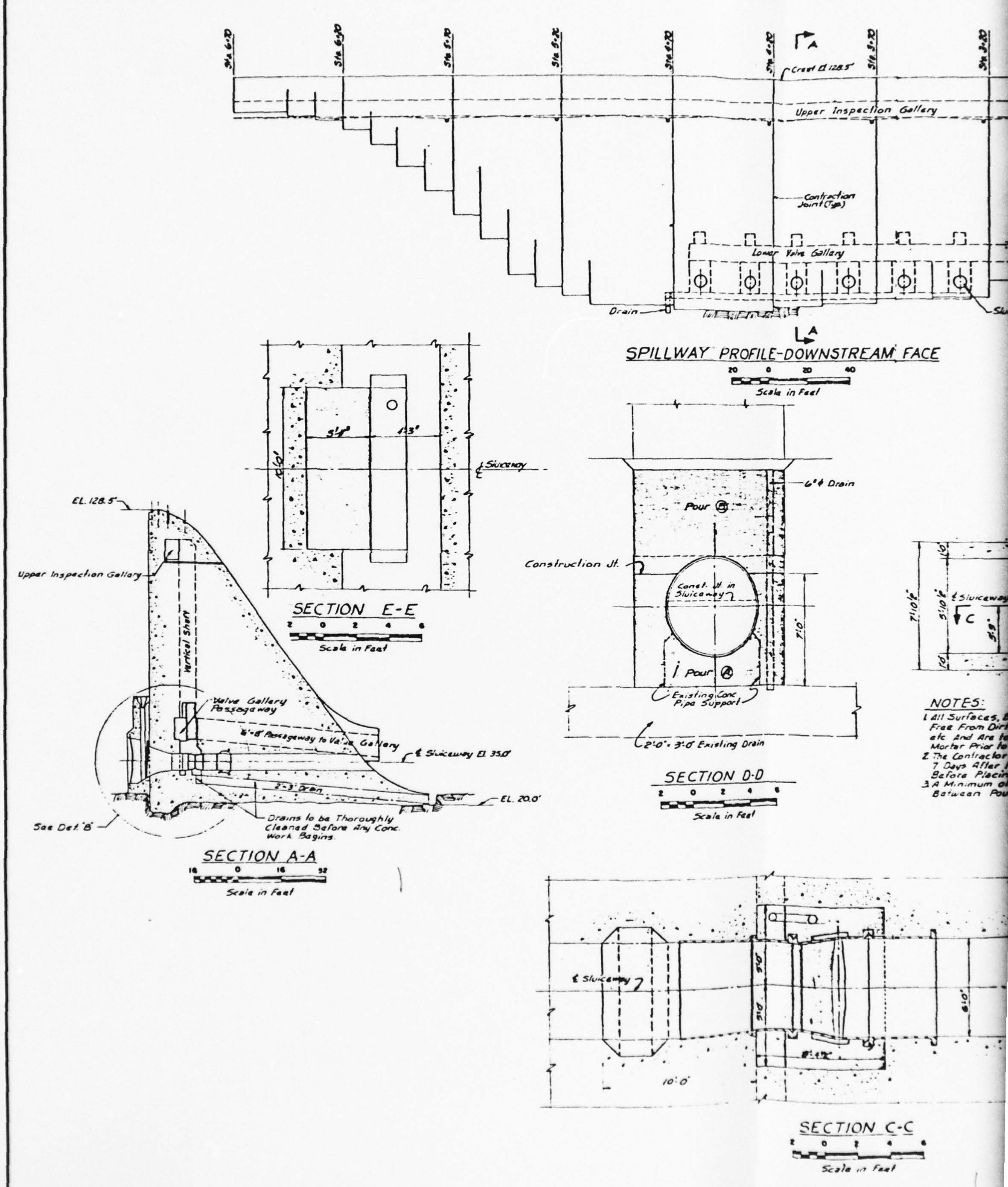
Point	1	2	3	4	5	6	7	8	9
Coordinate X	16.7	17.7	18.7	19.7	20.7	21.7	22.7	23.7	24.7
Coordinate Y	0.16	0.27	0.56	1.00	1.60	2.33	3.20	4.20	5.35

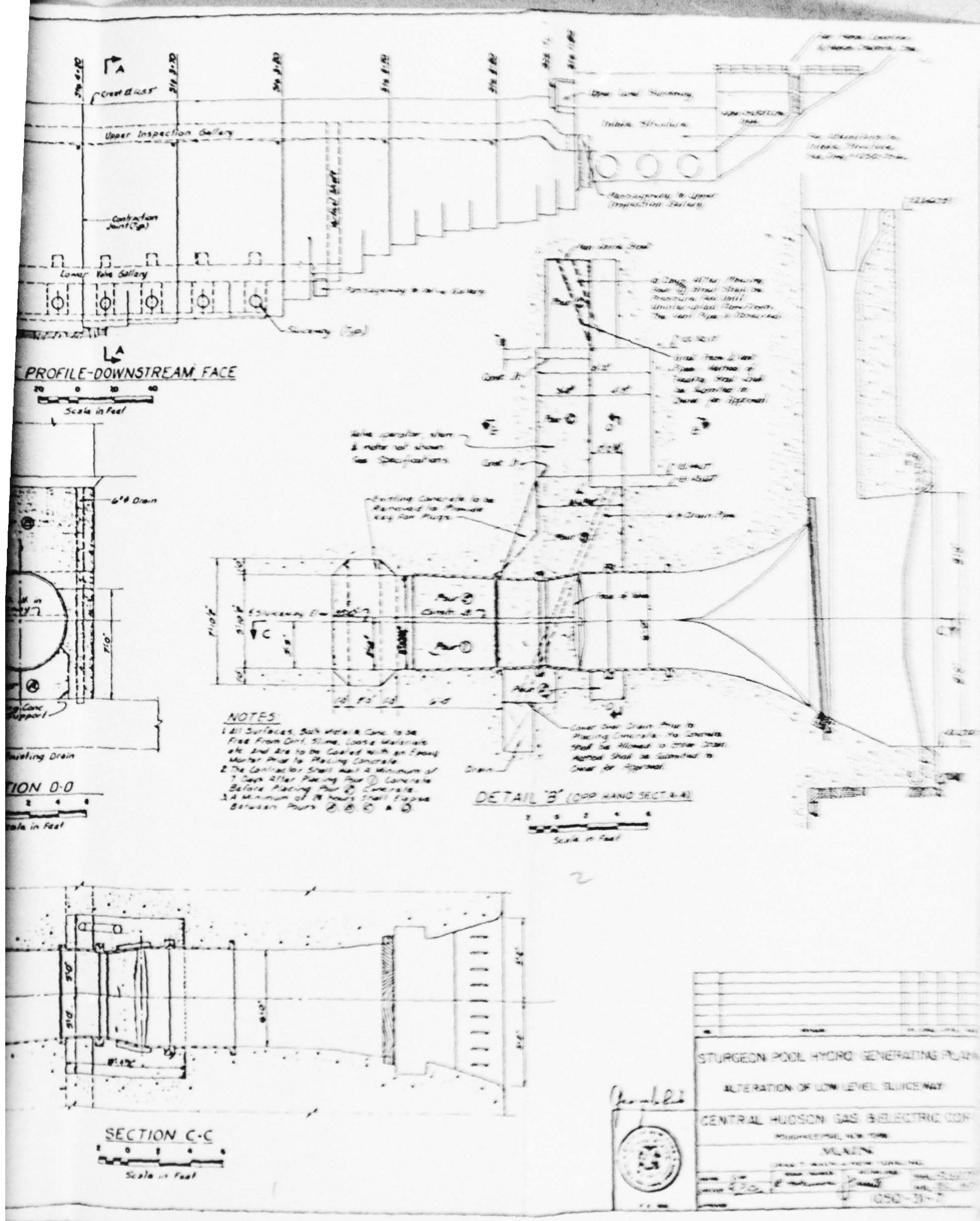

Granite facing
CRANE RAIL REMOVAL
Scale in feet
66 K.V. STEEL TOWER LINE
(Will remain Energized)
PROJECT LOCATION MAP
LAKE ONTARIO
LAKE ERIE
Buffalo
NEW YORK
Albany
Poughkeepsie
STURGEON POOL HYDRO
New York
(See Alteration Details on this Sheet)
NEW OVERFLOW DAM
TOWER (TYP.)
PROPERTY LINE
PENSTOCKS
POWERHOUSE
GARAGE
OUTDOOR SUBSTATION
(Will remain in Service Energized)
WOLLENHAUER SERVICE LINE
HIGH FALLS LINE
ROAD TO ROSENDALE
Pipe cap
30° Threaded elbow
and washers - galvanized (typ)
1" Marine Plywood
Galvanized Steel pipe
Top of existing wall
Drill hole for Extra Strong (Schedule 80) pipe sleeve and grout sleeve in place.
Caulk
DETAILS OF SPLASHBOARDS
Scale in feet
Galvanized steel railing to OSHA standards
DETAILS OF HANDRAILING
NOTES:
1. The location of the gate frame bolt anchors, installation details and concrete surface preparation shall be as recommended by the Gate manufacturer and as directed by the Owner.
2. The height of the timber operator's platform shall be determined by the owner after receiving gate manufacturers drawings and specifications.
3. The existing railing on the deck shall be removed and reinstalled on rebuilt slab.
4. The existing steel guides on the upstream face of the Upper Level Sluiceway are to be removed.
5. Concrete cover for all reinforcement
6. Slab designed for a 250 lbs/ft² Live Load.
7. Before placing new concrete clean of all loose and objectionable material. Then prepare surface with
SECTION C-C
SECTION D-D
See Note 6
These surfaces to have a steel trowel finish and then painted with asphalt to prevent bond
STURGEON POOL HYDRO GENERATING PLANT
LOCATION PLAN
AND
ALTERATION OF UPPER LEVEL SLUICEWAY
CENTRAL HUDSON GAS & ELECTRIC CORP.
POUGHKEEPSIE, NEW YORK
MAIN
CHAS T MAIN of NEW YORK, INC.
1050-31-5



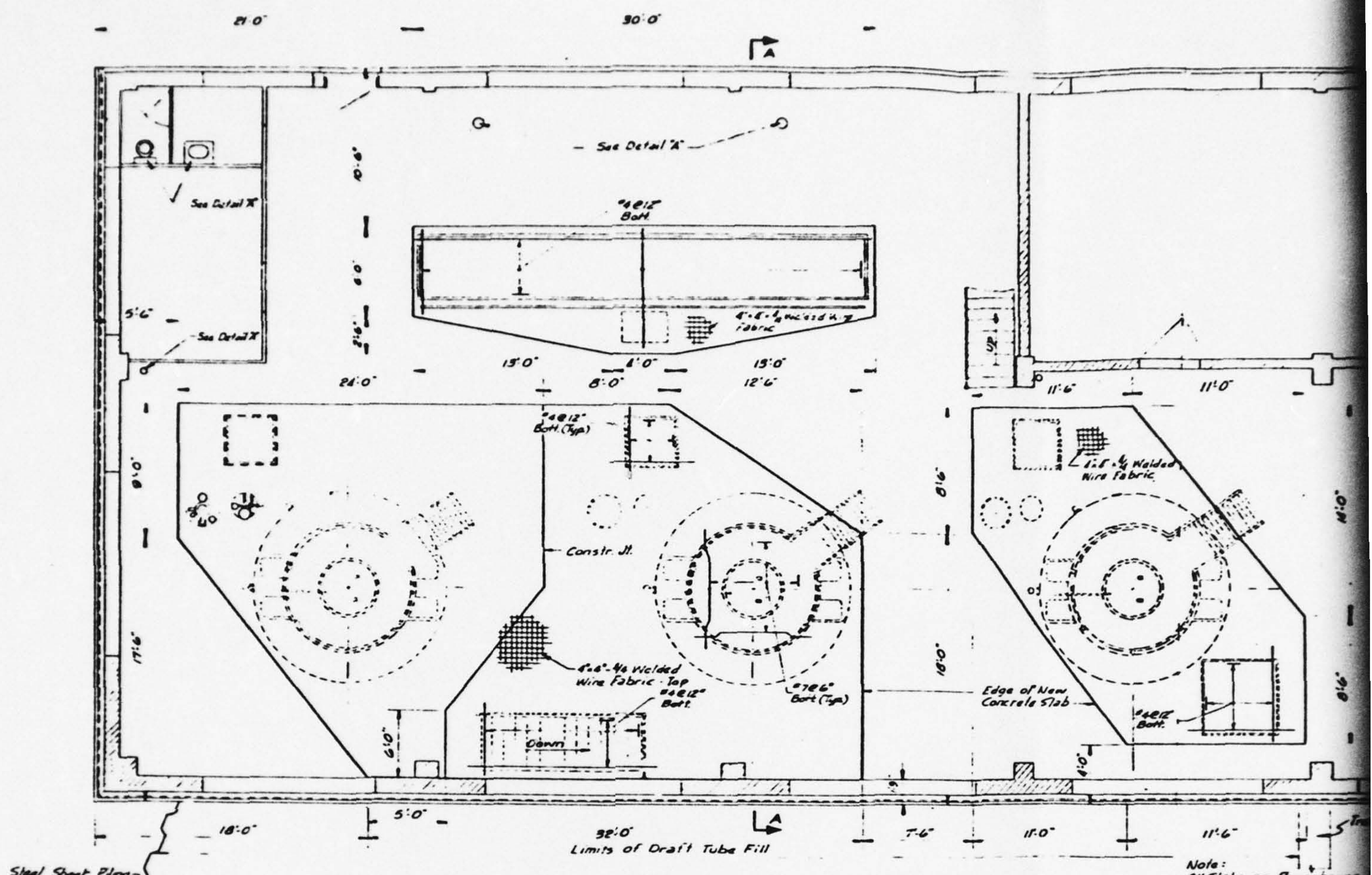
℄ PENSTOCK #1
℄ PENSTOCK #2
℄ PENSTOCK #3
Base Line of Dam
Stop Log Slots
Intake Screens (Typ)
PASSAGEWAY TO DAM
Gate Slot
2'x3' Air Vent (Typ)
Transition Section (Typ)
10'-0" Dia (Typ)
EL. 103.0'
EL. 98.0' (℄ of Penstock)
PLAN
Scale in Feet
Remove Covers, Thoroughly Clean Covers & Frames, Then Seal & Bed Covers in Epoxy Grout.
Back Up Opening with Back Fill
3' Wide - 6'-0" High Single Gate
Intake Structure
Non Overflow Dam
3½' Steel Post 6' High Chain Link Fence with Barbed Wire. All Fence Material to be Galvanized.
FENCE LOCATION PLAN
Scale in Feet
Continuous waterstop (Typ)
Remove 3'-9" of existing steel liner or amount as directed by Owner to facilitate placement of Concrete Plug. (Typ)
3 Strand Barbed Wire
1⅝" Top Rail (Typ)
1⅝" Brace Rail (Typ)
Tension Wire (Typ)
Note: Attach Fence Poles to Side of Dam with 2 Brackets
POST DETAIL
Scale in Feet
Penstock to be cut off See Detail "B"
PENSTOCK
See Detail "B"
End of Steel Penstock
26'-0"
Construction Jt.
Limits of 6" Conc Slab See Section A-A For Reinf Details
Manhole Covers See Section A-A (Typ)
℄ PENSTOCK
Preformed Corner Waterstop (Typ)
Continuous Waterstop
Pour ②
Constr. Jt.
Preformed 'T' Section Waterstop (Typ)
Pour ①
10'-0"
SECTION B-B
(TYP. PENSTOCK)
Scale in Feet
Existing Granite Block Facing
Concrete Block Masonry to be Laid To Match Granite Facing Line
℄ Penstock
Concrete Block
Existing Steel Penstocks to be cut off As Shown. Openings To be Blocked off With Concrete Block Masonry
Existing Concrete to be cut off at this point
1-2"x12" Water Drain per Penstock With Galvanized Rodent Screen
DETAIL "B"
Scale in Feet


FENCE LOCATION PLAN
Scale in Feet
POST DETAIL
Scale in Feet
SECTION A-A
(TYP. PENSTOCK EXCEPT AS NOTED)
Scale in Feet
NOTES:
1. The Contractor Shall Wait A Minimum of 7 Days After Placing Pour ① Concrete Before Placing Pour ② Concrete.
2. Surfaces of Intakes to be Cleaned Free From Dust And Loose Material Before Placing Concrete.
Remove Covers, Thoroughly Clean Covers & Frames, Then Seat & Bed Covers in Epoxy Grout
4"ϕ Vent Pipes, See Det. D (Typ.)
Base Line of Dam
Crest El. 128.5'
Air vent
PASSAGEWAY TO DAM
Concrete Plug
End of Steel Penstock
See Detail "B"
Remove Intake Screens. Support Members to remain in place.
DETAIL D
See Plan below For Location of Vent Pipes.
4"ϕ Galvanized Schedule 40 Pipe
Scale in Feet
6" P.V.C. Waterstops, 3/8 Thick. Contractor Shall Furnish Engineer With his Proposed Method of Supporting His Waterstop During Concreting of The Plugs
Cut existing Concrete in a Continuous Groove 6" Deep, 1½" Wide. Place Continuous P.V.C. Waterstop With Concrete, Comprised of Mortar Mixed With Epoxy.
DETAIL A
N.T.S.
℄ Penstock # 2
68'-0"
26'-0"
20'-0"
22'-0"
Construction Jt.
4"ϕ Gooseneck Vent Pipes Schedule 40 Cast into Slab (Typ.)
Construction Joint
Manhole Covers to be Grouted See Section A-A (Typ.)
PLAN @ EL. 141.5'
Scale in Feet
STURGEON POOL HYDRO GENERATING PLANT
ALTERATION OF INTAKE STRUCTURE
CENTRAL HUDSON GAS & ELECTRIC CORP.
POUGHKEEPSIE, NEW YORK
MAIN
CHAS. T. MAIN — NEW YORK INC.
1050-31-6

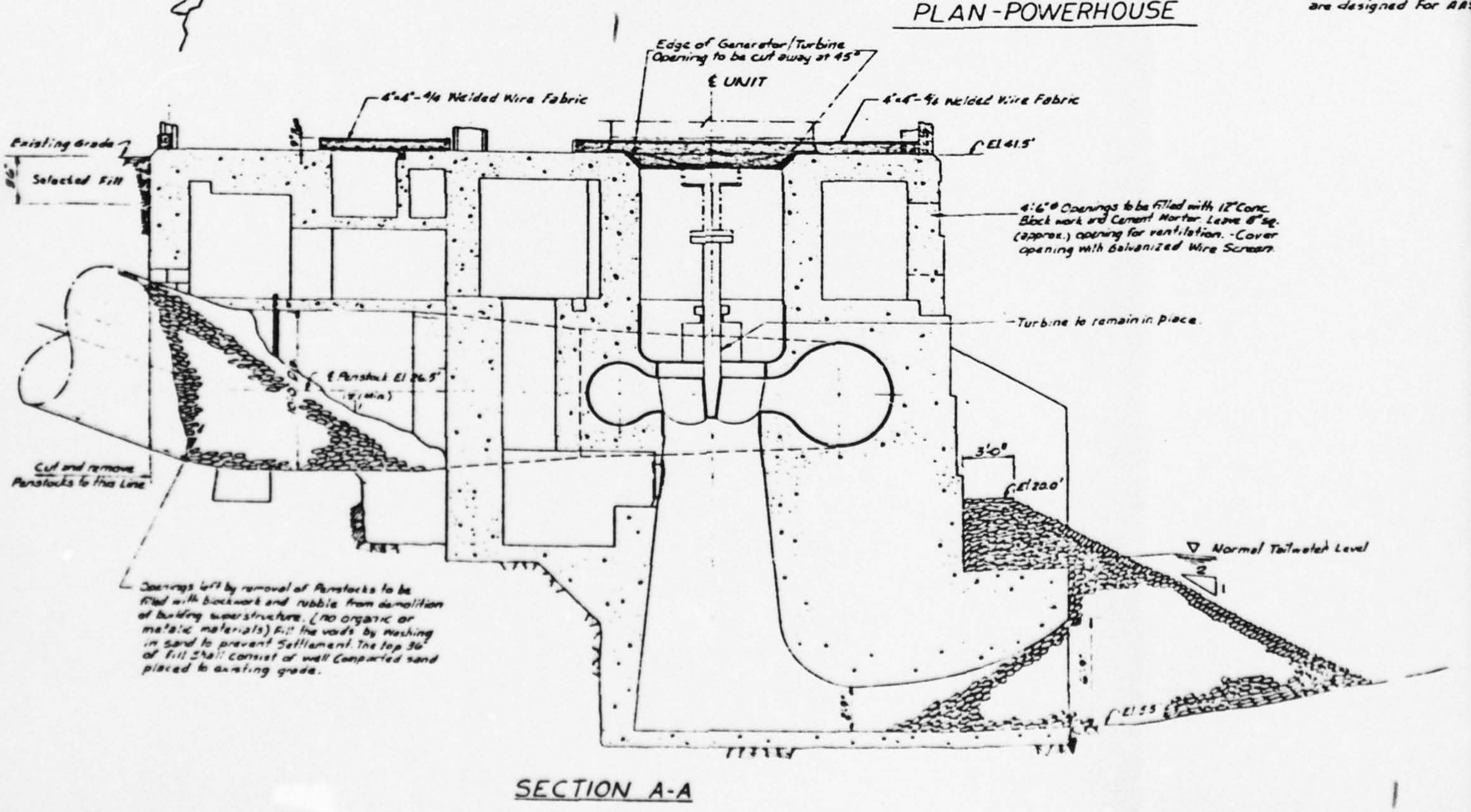

Crest El 128.5'
Upper Inspection Gallery
Contraction Joint (Typ)
Lower Valve Gallery
Drain
SPILLWAY PROFILE-DOWNSTREAM FACE
Scale in Feet
Sluiceway
SECTION E-E
Scale in Feet
6"φ Drain
Pour B
Construction Jt.
Const. Jt. in Sluiceway
7'0"
Pour A
Existing Conc Pipe Support
2'0" x 3'0" Existing Drain
SECTION D-D
Scale in Feet
EL. 128.5'
Upper Inspection Gallery
Vertical Shaft
Valve Gallery Passageway
6'-8" Passageway to Valve Gallery
℄ Sluiceway El 35.0'
2'-3' Drain
EL. 20.0'
See Det. 'B'
Drains to be Thoroughly Cleaned Before Any Conc. Work Begins.
SECTION A-A
Scale in Feet
7'10"
5'10"
℄ Sluiceway
NOTES:
℄ Sluiceway
10'0"
6'0"
SECTION C-C
Scale in Feet


Crest El.
Upper Inspection Gallery
Contraction Joint (Typ)
Lower Valve Gallery
Sluiceway (Typ)
PROFILE-DOWNSTREAM FACE
Scale in Feet
Intake Structure
Passageway to Upper Inspection Gallery
6" Drain
Existing Drain
SECTION D-D
Scale in Feet
Valve operator, stem & motor not shown. See Specifications
Existing Concrete to be Removed to Provide Key for Plugs
Sluiceway
NOTES
1. All Surfaces, Both Metal & Conc. to be Free From Dirt, Slime, Loose Materials etc. and Are to be Coated with an Epoxy Mortar Prior to Placing Concrete.
2. The Contractor Shall Wait A Minimum of 7 Days After Placing Pour 1 Concrete Before Placing Pour 2 Concrete.
3. A Minimum of 24 hours Shall Elapse Between Pours
Drain
DETAIL "B" (OPP HAND SECT A-A)
Scale in Feet
SECTION C-C
Scale in Feet
STURGEON POOL HYDRO GENERATING PLAN
ALTERATION OF LOW LEVEL SLUICEWAY
CENTRAL HUDSON GAS & ELECTRIC COR
POUGHKEEPSIE, NEW YORK
MAIN
1050-31-7

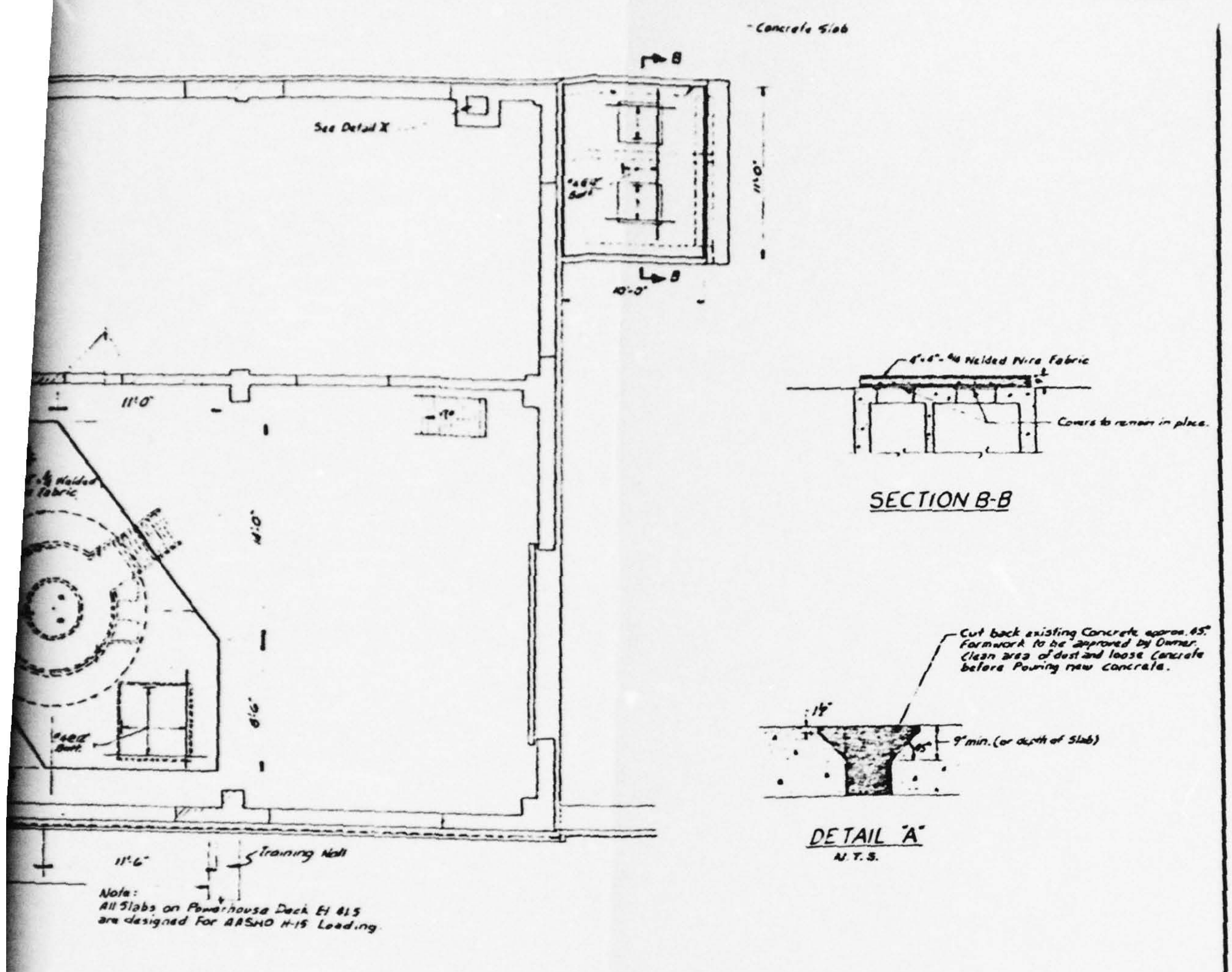
2



PLAN-POWERHOUSE



SECTION A-A



NOTES:

1. The Superstructure, Interior Walls, Partitions, Generators, and all Equipment Shall be Removed Above Elevation 415 as Required by The Specifications.
2. Holes Spalls, Chips or small Cracks left in The Slab Shall be Filled With Cement Mortar & Epoxy.
3. A 2" Min Cover is Required Over all Reinforcement.
4. The Manhole or Trench Covers Shall Remain in Place Where Covered by The New 9" Concrete Slab.

STURGEON POOL HYDRO GENERATING PLANT

ALTERATION OF POWERHOUSE

CENTRAL HUDSON GAS & ELECTRIC CORP.

POUGHKEEPSIE, NEW YORK

MAIN

CHAS. T. MAIN of NEW YORK INC.

SCALE AS NOTED

DATE DEC 1973

1050-31-8

CENTRAL HUDSON GAS & ELECTRIC CORPORATION

Sturgeon Pool Dam Structural Evaluation Phase I

May 1978

ACRES

ACRES AMERICAN INCORPORATED
Buffalo, New York 14202

June 15, 1978
P5006.01

Central Hudson Gas and
Electric Corporation
284 South Avenue
Poughkeepsie, New York 12602

Attn: Mr. Peter J. Rimsa
Production Operations
Engineer

Gentlemen:

Sturgeon Pool Dam
Structural Evaluation - Phase 1

We are pleased to submit six copies of our final report on the Phase 1 structural evaluation of the Sturgeon Pool Dam. The report describes our site inspection and office studies, together with our recommendations with respect to work required for subsequent phases of the evaluation. Your comments on the draft report received during our recent telephone conversations have been incorporated in this final report.

We appreciate the opportunity of preparing this Phase 1 report as well as your assistance in undertaking the evaluation.

Very truly yours,

M.R. Vanderburgh
Executive Engineer

MRV:adh

ACRES AMERICAN INCORPORATED

Consulting Engineers
The Liberty Bank Building, Main at Court
Buffalo, New York 14202

Telephone 716-853-7525

CENTRAL HUDSON GAS &
ELECTRIC CORPORATION

STURGEON POOL DAM
STRUCTURAL EVALUATION
PHASE I

MAY 1978

ACRES AMERICAN INCORPORATED
BUFFALO, NEW YORK 14202

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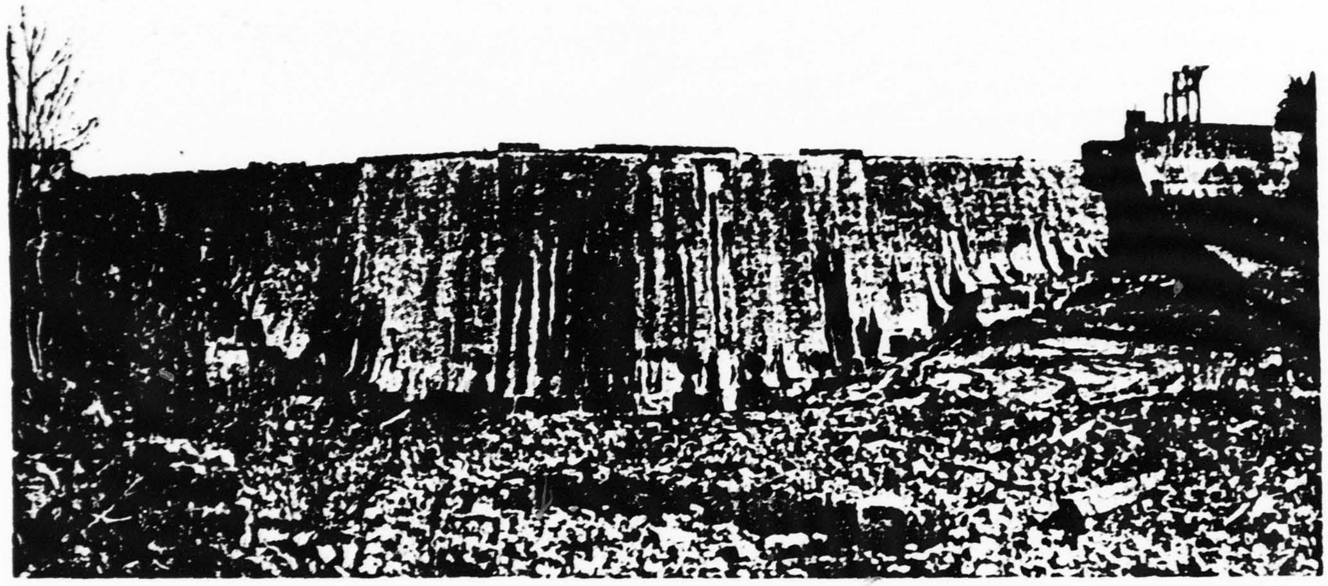

STURGEON POOL DAM

1.0 INTRODUCTION

This report presents the results of the Phase I evaluation of Sturgeon Pool Dam. This evaluation program described in Acres' proposal dated February 26, 1978, was authorized by Central Hudson Gas and Electric Corporation in a letter dated March 21, 1978.

The objective of this Phase 1 program was to:

(a) Undertake a structural inspection of the dam and a geological examination of the damsite area;

(b) Perform a stability analysis of the dam;

(c) Submit conclusions and recommendations to Central Hudson Gas and Electric regarding the condition of the dam and any on-going work which may be required.

The structural inspection and geological examination was completed on April 27, 1978. The structural evaluation was made on the basis of this site examination, supplemented by drawings and reports supplied by Central Hudson Gas and Electric. The results of the concrete coring and testing program undertaken by the Thompson & Lichtner Company, Incorporated, were also used in the evaluation.

A brief outline of the Phase 1 evaluation program, together with the main conclusions and recommendations, are presented in Section 2. Section 3 contains a geological description of the site area and the dam foundation on the basis of the geological walkover survey. In Section 4, the observations made during the dam inspection are included, together with comments on the importance of specific features which may represent potentially adverse conditions. Section 5 describes the methodology and results of the stability analysis which was performed on the intake, overflow, and non-overflow sections of the dam. Recommended factors of safety are presented and compared to actual factors of safety computed in the analysis. Section 6 presents the conclusions reached as a result of the Phase 1 inspection and analysis, and recommendations for further work.

2.0 SUMMARY

The Sturgeon Pool Dam is a concrete gravity structure which provides a storage pond of approximately 2,000 acre-feet for Central Hudson Gas and Electric Corporation's 15,000 KW Sturgeon Pool hydroelectric plant on the Walkill River, near Rosendale, New York.

For the purpose of the evaluation, three basic components of the dam were considered. The overflow section of the dam has a maximum height of 104 feet, and a crest length of 490 feet. On the north abutment of the river valley the overflow section adjoins the non-overflow section approximately 60 feet long. On the southern side of the valley the upper level sluiceway and intake structure for the penstocks are located.

The geological examination indicated the dam is founded on interbedded, highly folded strata of greywacke, siltstone and shale. Overall, the rock mass appears to exhibit a fairly high compressive strength and is relatively impermeable. The stability of the rock mass is controlled by bedding planes in the strata and jointing in the greywacke.

The structural examination did not indicate any obvious structural deficiencies. Seepage was observed at several points on the downstream face. A flow of water under pressure was observed on the dam at two locations near the right abutment.

The dam was analyzed to determine its relative safety against overturning and sliding. The results of the analysis indicated that the overflow section of the dam has an adequate factor of safety against overturning under normal operating conditions, but the factor of safety against overturning during extreme flood conditions was slightly less than normally accepted limits. The factors of safety against sliding were found to be extremely sensitive to the parameters assigned to the foundation rock, and the stability against sliding cannot be confirmed until more definitive data regarding the shearing resistance of the foundation is obtained. The analysis indicated that reduction in hydrostatic uplift in the foundation rock by the use of pressure relief drains drilled into the foundation would have a significant beneficial effect on stability.

The results of the concrete coring test program indicate the possibility that some zones of weak concrete exist in the dam, and stress levels in the structure may exceed allowable concrete strengths in these zones if they in fact do exist.

The main conclusions reached as a result of this Phase 1 evaluation are as follows:

- the stability of the dam with respect to sliding should be confirmed by field sampling and laboratory testing of the foundation rock to determine its overall frictional resistance

- subject to confirmation of adequate frictional resistance of the foundation rocks, the stability of the non-overflow and intake sections of the dam are acceptable

- all conditions of stability can be improved by installing pressure relief drain holes in the foundation

- the concrete coring and testing program should be extended to other areas of the dam in view of some of the poor quality core samples previously recovered

Our recommendations for further work are as follows:

- install 4 or 5 NX size exploratory drill holes from the lower gallery and from the downstream toe of the dam into the rock foundation to examine the foundation strata and to provide rock cores for further testing. These holes should be of sufficient depth to determine rock stratigraphy accurately

- install piezometers in the drill holes to measure the actual hydrostatic uplift pressures on the base of the structure

- perform a limited laboratory test program to determine representative values of angle of shearing resistance (Ø) and cohesion (c) in order to confirm the sliding resistance of the dam foundation both on rock to rock and rock to concrete interfaces

- extend the concrete coring and testing program to the lower gallery and at the toe of the dam to determine the compressive strength of the concrete near the base of the dam

- perform a more detailed analysis to determine the distribution of stress levels in the dam to compare with actual compressive strength test results obtained above

- areas of significant flow through construction joints in the dam or from the toe of the dam should be carefully mapped, and the practicality of sealing the construction joints by pressure grouting should be studied by attempting a field test. In areas where high flows are coming from behind the granite blocks near the face drain, holes should be drilled through the blocks. Sealing at the exit point of the flow should not be attempted

- the granite blocks on the downstream face of the overflow structure and in the crest should be carefully checked and reseated using mortar and epoxy resin where necessary to minimize further deterioration of the underlying concrete

CENTRAL HUDSON GAS & ELECTRIC CORPORATION
POUGHKEEPSIE, N.Y. 12602

HENRY L. WALKER
VICE PRESIDENT-PRODUCTION

(914) 452-2000

August 21, 1978

Mr. Cameron R. Mock
L. Robert Kimball Associates
615 West Highland Avenue
Ebanburg, PA 15931

Dear Mr. Mock:

This letter grants our authorization for Robert L. Kimball and Associates to conduct a federal Phase 1 inspection of Sturgeon Pool Dam as requested in your August 16, 1978 letter to our Mr. Donald Otis.

Enclosed to aid you in your inspection, are the following documents:

1. Central Hudson Gas & Electric Corporation Sturgeon Pool Hydro Generating Plant Report on Proposed Plant Retirement Considerations prepared by Chas. T. Main of New York, Inc.

2. Sturgeon Pool Dam Structural Evaluation Phase I prepared by Acres American.

3. Draft of Report on Restoration and Repair of Sturgeon Pool Dam prepared by Chas. T. Main of New York, Inc.

Please contact Mr. Donald Otis for any further assistance you may require and to arrange for one of our staff to accompany you during your field inspection.

Very truly yours,

Henry L. Walker

HLW;paw
Enclosures

- depending on the results of the drilling program to measure hydrostatic uplift on the base of the dam, and the results of the laboratory tests, a program should be formulated for drilling of vertical drain holes from the lower gallery of the dam to provide effective reduction in the uplift pressures. Normally these drain holes should be drilled into the rock approximately one half the height of the dam and spaced 10 foot intervals. Near the ends of the gallery they should be fanned on a plane parallel to the longitudinal axis of the dam to effect coverage of the adjacent portions of the foundation

- the feasibility of replacing the areas of deteriorated granite blocks on the crest of the dam with a high strength concrete bonded to the underlying existing concrete should be reviewed

3 - GEOTECHNICAL

3.1 - General

The Sturgeon Pool Dam is underlain by an alternating series of interbedded greywacke, siltstone and shale known as the Austin Glen member. The rock mass has a medium to high compressive strength. The greywacke is generally brittle and blocky, and the joints in the greywacke generally control the stability of the rock mass. The overall rock foundation is relatively impermeable, and seepage is generally confined to joints in the greywacke. The rock strata in the area has undergone significant folding. On the left abutment the beds dip upsteam, favoring dam stability; on the right abutment the intake structure and non-overflow sections are founded on beds which are horizontal, or dip slightly downstream.

A more detailed description of the rock types, rock structure and engineering properties is given in the following paragraphs.

3.2 - Rock Description

The rock strata comprising the dam foundation consists of Ordovician age (460 million years) interbedded greywacke, siltstone and shale of the Austin Glen member of the Normanskill formation. The greywacke is medium gray, fine to coarse grained, massive and hard. Microscopic analysis shows the greywacke containing angular grains of quartz, some rock fragments and minor amounts of feldspar and muscovite in a clay matrix with calcarous cement. Sedimentary structures such as sole marks, flute casts, load casts, ripple marks, slump structures and cross-bedding are evident.

The greywacke grades upward into medium gray siltstone, which, in turn, grades into gray shale. The thickness of this greywacke-siltstone-shale series varies typically from several inches to 3 feet. The shale and siltstone display a poorly developed bedding plane cleavage and occasional intricate flow structures.

3.3 - Rock Structure

Bedrock is exposed along the right abutment of the dam and along the stream bed downstream of the dam. The bedrock surface is competent with no weathered rock exposed at the surface.

Structurally, the Austin Glen greywacke in this area has been folded into a series of low amplitude open anticlinal and synclinal structures which strike N 45-55 E with fold axes plunging approximately 15° NE and axial planes dipping at approximately 20° to the northwest. Plate 4 is a bedrock structural map of the Sturgeon Pool Dam area. The

longitudinal axis of the dam foundation lies slightly askew of the regional structural trend, cross-cutting a syncline at a N 35° E orientation. The northeast half of the downstream dam foundation is founded on a steeply dipping southwest limb on an asymmetric syncline. Bedding on this limb dips at an angle of 55° NE. The southwest dam foundation is founded on the more gently dipping (10 to 15° NW) northeast fold limb.

Jointing on the Austin Glen member is lithologically controlled. While joints in the shale and siltstone are confined primarily to bedding planes, the greywacke is cut by regional joint sets. This phenomenon is apparently the result of the inability of the more plastic shale to sustain open spaces once the stress system which formed the regional joint sets was removed. Two prominent regional joint sets are evident in the greywacke: N 40° E, 85° SE and N 45° W, 75° SW. The joints are generally spaced between 2 and 3 feet apart.

A small fault striking N 60° E with apparent right lateral displacement was mapped along the right dam abutment. This fault, which is traceable for approximately 30 feet, disappears beneath the masonry wall on the northeast fault trace and into a shale bed on its southeast trace. Faults of similar nature are common within the Austin Glen, and have been interpreted as being formed during the early rock deformation (syndepositional). This fracture, which was very limited in extent, was examined and found to be generally healed with no evidence of excessive weathering along the fault plane.

Several small groundwater seeps were observed along joints and bedding planes on the right abutment. These seeps were considered to be minor and showed no evidence of excessive flow. A seep was also noted at the toe of the dam.

3.4 - Rock Properties

Boring and rock tests [6] performed on the Austin Glen greywacke in the Hudson Valley show the rock to be of high quality with RQD generally greater than 75 percent. Rock Quality Designation is an index describing the quality of the rock mass. It is a ratio of the sum of the lengths of intact pieces of core greater than ten centimeters in length to the length of core advance. Unconfined compressive strengths range from 8,600 psi to 25,800 psi with the average values for the unconfined compressive strengths, Young's modulus and Poisson's ratio to be 18,000 psi, 4.0×10^6 psi and 0.19, respectively. The average unit weight is 169 pcf.

As a whole, the rock is in the medium to high strength range. Due to the irregular, interbedded nature of the rock strata, average strength properties appear to be the most appropriate when considering the in-situ rock mass.

The stability of rock slopes within the Austin Glen greywacke is generally controlled by discontinuities such as joints in the greywacke,

and the bedding plane surfaces of the shale and siltstone. Direct shear tests performed on natural bedding joints in shale and siltstone from NX cored rock samples of the Austin Glen greywacke gave angles of shearing resistance (Ø) ranging from 20.8^{o} to 30.3^{o}.

4 - DAM INSPECTION

4.1 - Introduction

In accordance with Appendix D of "Recommended Guidelines for Safety Inspection of Dams"[4] published by the Office of the Chief of Engineers, Department of the Army, the Sturgeon Pool Dam would be classified as intermediate in size with a high hazard classification, since there is a high degree of downstream development. The hazard classification is only developed to describe the potential loss associated with a dam should a failure occur. High hazard classification indicates that "more than a few" lives would be lost and "extensive" economic loss would result.

A structural inspection of Sturgeon Pool Dam was conducted in April 27, 1978, to examine present conditions which might affect the safety of the dam. The following is a summary of this inspection visit.

4.2 - Exterior

The downstream face of the overflow section of the dam is surfaced with granite blocks which are infilled with grout or mortar. Most of the lower areas of the downstream face are in reasonably good condition with only minor misalignment of the granite blocks. Near the crest of the dam, however, several areas of granite block surfacing have been removed due to weathering and ice action. In these areas the underlying concrete is exposed and has crumbled to some extent due to weathering action.

At the time this dam was built (1922) the degree of control on concrete quality was not as high as today's standards. The use of entrained air in concrete as a protection device against the freeze-thaw cycle was not common. Thus, a high degree of concrete weathering in exposed areas would not be unusual.

To prevent water pressure build-up, a collector drain system below the granite blocks discharges water near the toe of the dam. However, in one area near the right abutment, water is apparently accumulating below the granite blocks and is discharging under pressure which is apparent from the size of the jet coming through the blocks near the right abutment.

The drains which discharge water from the upper gallery of the dam to the downstream face are apparently operating and the discharge of water is evident at several locations on the face. Water under pressure is discharging through the granite blocks near the right abutment at a high level (above the upper gallery). Apparently there is an accumulation of water below the granite blocks from a source other than the gallery and, presumably, it is coming through a vertical contraction joint.

4.3 - Structural Integrity

There are no visible displacements on the overflow section of the dam at the expansion joints and, other than loose granite blocks, the dam generally appears to be in good condition. One vertical crack does exist however, near the junction of the overflow section of the dam and the intake. This crack has a small amount of displacement across it and appears to be a result of differential movement between the intake and the overflow section. It does not appear that this crack, although visible on the downstream face, has any detrimental effect on the integrity of the dam.

The non-overflow section of the dam appears to be in good condition.

4.4 - Upper Gallery

A longitudinal section (Plate 3) shows the position of the upper gallery in the dam. A visual inspection of this gallery indicates that the concrete is generally in good condition with the exception of some spalling near the contraction joints. In some cases, this may have been caused by attempts to seal the joint against seepage which resulted in a "blowing off" of both the patch and some adjacent intact concrete.

Water inflows to the gallery were not excessive on the day the inspection was carried out. The maximum flow from any joint was estimated to be 20 gallons per minute.

According to Central Hudson personnel, this seepage is considerably higher during colder weather. It would appear that waterstops near the upstream face of the dam have deteriorated and are not very effective in reducing flow.

Several of the drains exiting from the upper gallery are plugged and do not carry water.

4.5 - Vertical Stairway

The stairway connecting the upper and lower galleries affords a good opportunity to view the concrete condition. There is no apparent seepage coming into this stairway and the condition of the concrete is generally good.

4.6 - Lower Gallery

The lower gallery, which is the valve passageway in the dam, is shown on Plate 3. The concrete condition in the lower gallery is good. There are areas of precipitate build-up in some locations, but these

do not have any significant effect on the structure.

The gate valves are operated from the lower gallery, however, these have not been operated for some time and it is not known if they will function properly.

4.7 - Upstream Face

It is understood that subsequent inspections of the upstream face during a partial reservoir drawdown by Central Hudson have revealed two basic features:

a. The granite blocks extend to 5 feet below the crest on the upstream face.

b. Deposits of silt were noted on the upstream face at some horizontal construction joints indicating flow through the joint and the possibility of a seepage path wide enough to permit movement of silt particles partially or completely through the dam.

5 - STABILITY AND STRESS ANALYSIS

5.1 - General

Stability analysis procedures for gravity dams involve consideration of sliding, buoyancy, overturning, and overstressing within the dam and its foundation[3].

Three cross sections were examined with respect to stability; these comprise:

a. the overflow section (typical of 490 feet in mid-section)

b. the intake structure

c. the non-overflow section (typical of approximately 45 feet at either end of the dam).

Plates 5, 6 and 7 show the cross-sections examined.

The analysis involves a two dimensional representative cross-section subjected to a variety of loading conditions. The loading conditions used in the Sturgeon Pool analysis are those recommended by the Army Corps of Engineers [5] and include:

F_1 - water load on the upstream face (lbs. per foot of dam)

F_2 - water load on the downstream face (lbs. per foot of dam)

W - weight of a one foot section (lbs.)

U - hydrostatic uplift pressures (lbs. per foot of dam)

I - ice load (lbs. per foot of dam)

S - silt load (lbs. per foot of dam)

EQ - seismic load (lbs. per foot of dam)

P - water pressure created by seismic load (lbs. per foot of dam)

The Sturgeon Pool Dam site is located in a Zone III earthquake zone[6]. This zone is associated with earthquake intensities of VI to VII (Modified Mercalli scale). The acceleration associated with an intensity VI earthquake, attentuated to a distance of 15 miles, is 0.06g.

The loading combinations assumed for analysis of each stability condition are listed below. An explanation of the notations used is given in Plates 8 and 9.

Normal Operation - Overflow Section

Buoyancy Safety Factor

- without earthquake

$$SF = \frac{W}{U}$$

- with earthquake

$$SF = \frac{W}{U+EQ}$$

Overturning Safety Factor

- without earthquake

$$SF = \frac{W(b-x)}{F_1(f_1) + U(b-u) + I(i) + S(s)}$$

- with earthquake

$$SF = \frac{W(b-x)}{F_1(f_1) + U(b-u) + S(s) + EQ(y) + P(p)}$$

Sliding Safety Factor

- without earthquake

$$SF = \frac{(W-U) \tan \emptyset}{F_1 + I + S}$$

- with earthquake

$$SF = \frac{(W-U) \tan \emptyset}{F_1 + S + EQ + P}$$

Flood Conditions - Overflow Section

Buoyancy Safety Factor

- without earthquake

$$SF = \frac{W}{U}$$

- with earthquake

$$SF = \frac{W}{U+EQ}$$

Overturning Safety Factor

- without earthquake

$$SF = \frac{W(b-x) + F_2(f_2)}{F_1(f_1) + S(s) + I(i) + U(b-u)}$$

- with earthquake

$$SF = \frac{W(b-x) + F_2(f_2)}{F_1(f_1) + S(s) + U(b-u) + EQ(y) + P(p)}$$

Sliding Safety Factor

- without earthquake

$$SF = \frac{(W-U)\tan \emptyset}{F_1 + S + I - F_2}$$

- with earthquake

$$SF = \frac{(W-U)\tan \emptyset}{F_1 + S + EQ + P - F_2}$$

In both the intake and non-overflow sections, the silt load was neglected since it is likely that water movement in these areas of the pond would not allow silt deposition.

The water level used for the analysis under flood condition was elevation 137.3 feet which was equivalent to the Standard Project Flood level developed during previous studies[2] and slightly higher than the maximum reported level of 135.8 feet in 1955. The effect of opening six sluice gates during flood conditions was examined. The change in headwater elevation due to the reduced flow volume passing over the dam was found to be less than one foot. The effective reduction of the headwater force on the dam is negligible.

Details of the mathematical assumptions used with regard to headwater, tailwater, ice, silt, seismic water pressures, earthquake, and uplift loads are found in Appendix A.

The allowable factors of safety for all conditions are listed in Table 5.1. These values are based on previous project works done by the engineer and are consistent with traditionally accepted values.

TABLE 5.1

Allowable Safety Factors

	Without Earthquake Load		*With Earthquake Load*	
	Normal Operation	Extreme Conditions	Normal Operation	Extreme Conditions
Buoyancy	1.2	1.1	1.1	1.1
Overturning	1.2	1.1	1.1	1.1
Sliding - with cohesion	4.0	2.67	1.5	1.1

5.2 - Overturning

The factor of safety for overturning is computed with respect to an axis of rotation about the toe of the dam (Part B) as shown on Plates 8 and 9. The safety factor represents a ratio of resisting moment (RM) to overturning moment (OM).

$$FS = \frac{\Sigma RM}{\Sigma OM}$$

The loading cases used on each section are more fully described in Section 5.1.

The results of the overturning analysis is given in Table 5.2.

TABLE 5.2

Factor of Safety Against Overturning*

	Without Earthquake		With Earthquake	
Section	Normal Operation	Flood Conditions	Normal Operation	Flood Conditions
Overflow	1.42	1.03	1.33	0.98
Intake	2.38	2.04	2.39	1.88
Non-Overflow	2.17	1.89	3.14	1.69

*Full uplift considered

Under flood conditions, with and without earthquake loads, the factors of safety for the overflow section fall below the normally acceptable minimum safety factor of 1.1. Given the extremely low probability that an earthquake will occur while the dam is simultaneously undergoing flood conditions, a factor of safety which is nearly 1.0 may be acceptable. The low factor of safety (1.03) for the overflow section under flood conditions could be improved by pressure relief drains in the foundation.

The problem of overturning could be worsened by undercutting of the downstream toe of the dam. An analysis of the overflow section was performed to check the effects of undercutting with a minimum of 10 feet to undercut assumed. The effect on the factor of safety with respect to overturning is shown in Table 5.3.

TABLE 5.3

Effect of Undercutting of Dam Toe on Factor of Safety Against Overturning

	Without Earthquake		With Earthquake	
Overflow Section	Normal Operation	Flood Condition	Normal Operation	Flood Condition
Without undercutting	1.42	1.03	1.33	0.98
With undercutting	1.33	0.98	1.24	0.96

It was not possible to determine the extent of undercutting at the toe during the site inspection visit. It is understood, however, that the area beneath the toe has been backfilled with concrete in recent years in order to control the problem of undercutting. Further investigation would be required to insure that sound conditions exist at the toe of the dam.

5.3 - Buoyancy

The factor of safety concerned with buoyancy is based on a ratio of weight to uplift as described by the equation below:

$$FS = \frac{\Sigma \text{ Vertical Forces acting down}}{\Sigma \text{ Vertical Forces acting up}}$$

Buoyancy is not usually a problem associated with concrete gravity dams as is reflected in the factors of safety listed in Table 5.4.

TABLE 5.4

Buoyancy Factors of Safety*

Section	Without Earthquake		With Earthquake	
	Normal Operation	Flood Conditions	Normal Operation	Flood Conditions
Overflow	2.60	1.40	2.06	1.27
Intake	3.48	2.96	2.87	2.38
Non-Overflow	4.78	3.14	3.73	2.65

* Includes full uplift consideration

5.4 - Sliding

The factor of safety against sliding is a ratio of the resistance at the rock-concrete interface to the net horizontal force.

$$FS = \frac{N \tan \phi + cA}{\Sigma H}$$

N = force normal to sliding plane

ϕ = angle of internal friction

c = cohesion intercept

A = area subject to cohesive shear

H = horizontal loads

The resistance offered by the rock to sliding can be divided into two parts, frictional resistance and cohesive shear resistance. The rock property which defines frictional resistance ϕ, depends on the composition of the particles and varies within a very small range for a given rock type. Tests performed on samples of a similar siltstone-shale-sandstone sequence from a site nearby[6] indicated ϕ values ranging between 20.8° and 30.3°. The average of the test results was 25° with a minimum value chosen at 23°[6].

The second component of resistance is reflected in the rock strength parameter cohesion, c. The value of c depends on the shape of the grains and degree of interlock on both sides of the sliding surface. The cohesion intercept, c, can be determined from various tests. In the absence of representative test results, opinion varies considerably as to an assumed value for cohesion. Traditional values of cohesion have ranged from 15

to 150 psi[4]. Deere[4] reports that "a preliminary estimate may be made by assuming that c is zero and that Ø for a smooth surface would be in the following ranges: 30° to 35° for hard, massive rocks of well-cemented or interlocking texture (sandstones, basalt, granite, limestone, etc); 25° to 30° for hard, shaly or schistose rocks (hard shale, slate, phyllite, mica schist); and 20° to 25° for softer laminated or schistose rocks (clay-shale, talc schist, chlorite schist)." Jones[10] suggests that "in areas where the weathering, shearing or altering has produced a soil, the long-term condition calls for a zero cohesion analysis.

With the foregoing in mind, a series of calculations of the factor of safety with respect to sliding was performed to reflect the change in assumptions with regard to the geology. In an analysis which neglects cohesion, the resulting factors of safety fall considerably below unity. The approach presents an unrealistic picture with regard to sliding stability.

The effect of cohesion on the factor of safety is substantial. Thus, the assumptions regarding this rock parameter are critical to the results obtained. Current practice for foundation preparation of concrete dams involves removal of all unsound rock as well as removal of soil and other deleterious matter to ensure good contact between the base of the dam and the rock foundation. Sturgeon Pool Dam was built in 1922 and records which describe the work which was performed are not available. To account for the lack of information and in the absence of any specific test results a conservative assumption on the value of the cohesion intercept should be used.

During the inspection of the dam in April, 1978, several leaking construction joints and water seepage at the bedding planes were observed. The evidence of seepage through the construction joints in the dam gives support to the possibility of seepage at the concrete-rock interface. This type of seepage could lead to erosion of the foundation rock which would reduce the effective cohesion.

In reviewing the stability of Sturgeon Pool Dam with respect to sliding the allowable factors of safety as listed in Section 5.1 were assumed. The cohesion intercept required of the rock to meet those factors of safety with an angle of interval friction of 23 degrees was determined. Table 5.5 lists the resulting values.

TABLE 5.5

Cohesion Required to Meet Allowable Factors of Safety*

Section	Without Earthquake: Normal Operation	Without Earthquake: Flood Condition	With Earthquake: Normal Operation	With Earthquake: Flood Condition
Overflow	89 psi	34 psi	66 psi	29 psi
Intake	15	3	13	2
Non-overflow	17	4	4	3

* $\emptyset = 25^{o}$

Further investigation of actual conditions at the concrete-rock interface are required to assure that the existing cohesion values are above the minimum listed in Table 5.5.

The dam is situated on a bedded shale-sandstone-mudstone synclinal structure. The central portions of the dam lie on beds which dip at approximately 45° in a direction favorable to stability against sliding. The intake structure and non-overflow sections on the southern end are founded on beds which are nearly horizontal, or possibly at a 5° dip unfavorable to sliding stability.

To evaluate the stability on horizontal layers, the weakest bed is assumed to act as a failure plane. Thus, the minimum value of the angle of interval friction is used. At Sturgeon Pool this was assumed to be $\emptyset$=23 degrees. In favorable dipping beds, the failure plane between the concrete and the rock crosses the beds. Thus, an average value of the friction coefficient, or a $\emptyset$ of 25 degrees, is appropriate. However, the change in the value of cohesion required to meet the allowable factors of safety when the angle of interval friction $\emptyset$, is varied from 23 to 25 degrees is negligible.

5.5 - Foundation Pressure Relief

In accordance with design practice at the time, Sturgeon Pool Dam was built without provision to reduce the uplift forces beneath the dam. The lower gallery, located near the upstream face, provides opportunity to locate a row of pressure relief holes to reduce uplift pressures in the foundation. An evaluation of the effect of pressure relief drains was made for the overflow section. Reducing the uplift increases the factor of safety for all modes of stability--buoyancy, overturning, and sliding.

The drains were assumed to be effective in reducing the uplift pressure by 1/3 of the head difference[3]. A linear distribution between the headwater and the drains and between the drains and the tailwater was used. The effect on the factors of safety for buoyancy, overturning, and sliding can be seen in Table 5.6.

TABLE 5.6

Factors of Safety with Reduced Uplift Pressure Considered

	Without Earthquake		With Earthquake	
Overflow Section	Normal Operation	Flood Condition	Normal Operation	Flood Condition
Buoyancy				
- full uplift	2.26	1.40	2.06	1.27
- reduced uplift	5.36	2.72	4.87	2.46
Overturning				
- full uplift	1.42	1.03	1.33	0.98
- reduced uplift	2.18	1.57	1.96	1.46
Sliding (Cohesion Required)				
- full uplift	89 psi	34 psi	66 psi	29 psi
- reduced uplift	82	26	60	20

5.6 - Dewatering the Intake

Special consideration with regard to stability must be given to the intake structure section if the power plant were shut down and the penstocks and intake structure were dewatered. The factors of safety for all three stability modes are listed in Table 5.7.

TABLE 5.7

Intake Structure Section Dewatered Safety Factors

	Without Earthquake		With Earthquake	
Intake Section	Normal Operation	Flood Condition	Normal Operation	Flood Condition
Buoyancy	2.28	2.04	2.01	1.82
Overturning	1.40	1.20	1.45	1.13
Sliding* (Cohesion Required)	29 psi	11 psi	19 psi	9 psi

* Ø = 25 degrees

5.7 CONCRETE STRESSES

Stresses in concrete gravity dams typically reach maximum values in compression at the downstream toe of the dam and maximum values in tension at the base of the upstream face of the dam. Typical concrete compressive strengths specified for mass concrete applications, such as dams, are in the range of 2000-3000 psi.

For stability purposes the maximum allowable compressive stress for concrete in the dam should be less 33 percent of the actual compressive strength for normal loading conditions, and 100 percent of specified for extreme loading combinations. In no case should the compressive stress exceed 1500 psi for normal loading conditions or 2250 psi for extreme loading combinations.

Tensile stresses should be avoided whenever practicable. Any tensile stresses which do exist should not exceed 150 psi for normal load conditions or 225 psi for extreme load conditions. If the concrete tensile strength is exceeded, the concrete section should be considered cracked in future structural evaluations.[15]

The results of the dam inspection performed April, 1978, indicated that the concrete generally appeared to be in good condition on visual inspection. This does not provide information on the remainder of the mass structural concrete which is the primary structural element of the dam.

Information provided by Central Hudson on concrete strength test results performed on concrete cores taken from the upper gallery indicated three samples which disintegrated or fell apart.

The strength of test cores indicates the strength of a structure and also reflects other properties such as materials used in the original mix design, durability, and watertightness of the concrete. Large variations in the test results necessarily require a higher average strength to provide an adequate factor of safety. Furthermore, excessive variations in quality always suggest the danger of inferior concrete.[11]

The quality of control may determine to a large extent the useful life of a structure. The strength of standard test specimens varies above and below an average and falls in some pattern of a normal probability curve. When there is good concrete, strength values vary little from average, and the curve is steep. With poor control, the values are spread laterally, and the curve is flattened. The radius of gyration of points about the center is called the coefficient of variation. The coefficient of variation is a statistical tool used to establish the amount of variation.

A high coefficient of variation indicates poor control, and a low coefficient indicates a good control. The maximum allowable coefficient of variation in concrete strengths of U.S. Bureau of Reclamation projects is 0.25; the value obtained for the concrete samples taken at Sturgeon Pool is 0.42.[11]

Criteria generally accepted by Bureau of Reclamation designers require that the strength of 80 percent of the test specimens be greater than the design strength. Available test results show that 78 percent of the strength values for the Sturgeon Pool Dam samples are greater than 2500 psi, the average strength of the samples is 2757 psi, however, three of the samples fell apart or disintegrated either before or upon application of load.

Factors of safety for concrete stresses are defined by the ratio of allowable stress to actual stress:

$$FS = \frac{\text{Allowable Stress}}{\text{Actual Stress}}$$

In every loading case examined the allowable stress was well above the factored actual stress. The lowest factors of safety occur for both tension and compression in the loading case with the following considerations:

- no pressure relief dams
- flood (extreme) condition
- with earthquake
- ogee section

In the above loading condition a tensile stress of 35 psi was found to exist at the base of the upstream face of the dam. The greatest compressive stress of 219 psi occurs at the downstream toe of the dam.

Normally, the above stresses would indicate an acceptable factor of safety; however, due to the poor test results from the concrete core samples, a dependable estimate of actual strength of the concrete is uncertain.

The above results are based on a simplified two-dimensional slice analysis of a typical section for each of the three different segments of the dam being analyzed. A more detailed three-dimensional principal stress analysis might indicate slightly higher stresses than those determined on a two-dimensional slice analysis.

Recognizing that the most critical stresses occur at the base of the dam, it is recommended that concrete cores be taken as close to the base section of the dam as possible, perhaps from the lower gallery. Appropriate tests should be performed in order to get specific information on the quality and strength of the concrete in this region of the dam. It is also advisable to do a more in-depth stress analysis of the dam in the form of a three-dimensional principal stress analysis or a finite element analysis.

5.8 - Bearing Capacity

Recent practice has been to evaluate bearing capacity of rock foundations in terms of compressive strength of the rock. One approach is to assign a bearing value equal to a percentage of the unconfined compressive strength, usually 10 to 20 percent, based on a judgement of the quality of the rock mass. At Sturgeon Pool the limiting value bearing capacity would be 1800 psi. With maximum compressive forces of 26 and 50 psi for normal and flood conditions, respectively, the bearing capacity of the dam foundation is well within acceptable limits.

6.0 CONCLUSIONS AND RECOMMENDATIONS

It is apparent that the Sturgeon Pool Dam is presently stable, as evidenced by its performance. Analyses of three sections of the dam were conducted. These comprised the overflow, the intake and non-overflow sections. The intake and non-overflow sections were found to be stable with respect to sliding, overturning, and buoyancy based on estimated parameters. Computed factors of safety for overturning of the overflow section of the dam based on estimated uplift conditions fall slightly below normally accepted limits. This condition, however, can be significantly improved by installing pressure relief drain holes in the foundation of the dam. Sliding stability analysis of this section based on assumed rock parameters gives relatively low factors of safety. Testing of the foundation rock is required to permit the use of more accurate parameters in the analysis.

Under extreme flood conditions, the factor of safety against overturning was computed to be 1.03, compared to the normal allowable value of 1.10. A safety factor near unity results when the effects of flood conditions and earthquake are taken together, a severe combination of loading.

Sliding stability is dependent on two rock strength parameters, internal friction and cohesive shear strength. In the absence of testing on rock samples from the dam foundation, conservative estimates of these parameters must be used. The internal angle of friction Ø, varies only slightly for a given rock type and the effect of small variation in values of Ø on the stability of the dam is not substantial.

The value of cohesion used in the analysis has a very significant effect on the stability of the structure and the assumptions regarding the rock parameter are therefore of considerable importance in this evaluation. Some investigators have suggested that in the absence of test results, a conservative assumption of zero cohesion should be used. This assumption would result in theoretical factors of safety well below unity for this structure under normal conditions, so that it is obvious that some value of cohesion must be considered in the analysis. It appears therefore that the only practical means of resolving this aspect of the evaluation is to obtain the required parameters by testing of representative samples of the foundation rock in direct shear.

Water pressure beneath the dam is a major influence on all modes of stability. The conventional assumption of a linear distrution between the headwater and tailwater elevations was used in the analysis. Actual uplift pressure may vary from this assumption, and can only be verified through the installation of piezometers in drill holes in the foundation. The influence of reducing the uplift pressures by installing a line of pressure relief drains from the lower gallery was investigated. The effect of this assumption was to significantly increase factors of safety for all modes of stability.

The stability of the intake during dewatering was analyzed. The resulting safety factors indicate that the structure will remain stable under this condition.

The results of the concrete coring and testing program indicated that some of the samples tested disintegrated before any measurable load could be applied. Extrapolating this failure rate to the more highly stressed lower area of the dam could result in zones where stress levels exceeded permissable concrete strengths.

Our recommendations for further work are as follows:

- Install 4 or 5 NX-size (3.0 inch) exploratory drill holes from the lower gallery and from the downstream toe of the dam into the rock foundation to examine the foundation strata and to provide rock cores for further testing. These holes should be of sufficient depth to give an adequate picture of the underlying rock stratigraphy.

- Install piezometers in the drill holes to measure the actual hydrostatic uplift pressures on the base of the structure;

- Perform a limited laboratory test program to determine representative values of angle of shearing resistance (Ø) and cohesion intercept (c) in order to confirm the sliding resistance of the dam foundation both on rock to rock and rock to concrete interfaces;

- Extend the concrete coring and testing program to the lower gallery and at the toe of the dam to determine the compressive strength of the concrete near the base of the dam;

- Perform a more detailed analysis to determine the distribution of stress levels in the dam to compare with actual compressive strength test results obtained above;

- Areas of significant flow through construction joints in the dam or from the toe of the dam should be carefully mapped, and the practicality of sealing the construction joints by pressure grouting should be studied by attempting a field test. In areas where high flows are coming from behind the granite blocks near the toe drains, holes should be drilled through the blocks. Sealing at the exit point of the flow should not be attempted;

- The granite blocks on the downstream face of the overflow structure and in the crest should be carefully checked and reseated using mortar and epoxy resin where necessary, to minimize further deterioration of the underlying concrete;

- Depending on the results of the drilling program to measure hydrostatic uplift on the base of the dam, and the results of the laboratory tests, a program should be formulated for drilling of drain holes from the lower gallery of the dam to provide effective reduction in the uplift pressure. Normally these drain holes should be drilled into the rock approximately one half the height of the dam and spaced at 10 foot intervals. Near the ends of the gallery they should be fanned out with a 15 degree spacing from vertical to horizontal.

REFERENCES

1 American Society for Testing and Materials, 1977 Annual Book of Standards; Part 14, ASTM; Easton, 1977.

2 Chas. T. Main of New York, Inc., Sturgeon Pool Hydro Generating Plant - Report on Proposed Plant Retirement Considerations, December, 1973. ✓

3 Creager, William P. and Joel D. Justin. Hydroelectric Handbook. John Wiley B. Sons, Inc.; New York, 1950.

4 Deere, Don U. "Dams on Rock Foundations - Some Design Questions". Rock Engineering for Foundations & Slopes. American Society of Civil Engineers; New York, 1976.

5 Department of the Army. Recommended Guidelines for Safety Inspection of Dams. Office of the Chief of Engineers; Washington.

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8 Hamel, James V., et al. "Mahoning Dam Foundation Re-Evaluation". Rock Engineering for Foundations & Slopes. American Society of Civil Engineers; New York, 1976.

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APPENDIX A

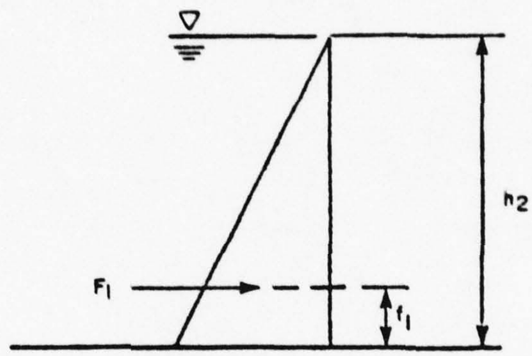
APPENDIX A

Load Condition Assumptions

Headwater Loads[3]

$F_1 = 1/2\ \gamma_w\ h_2^2$

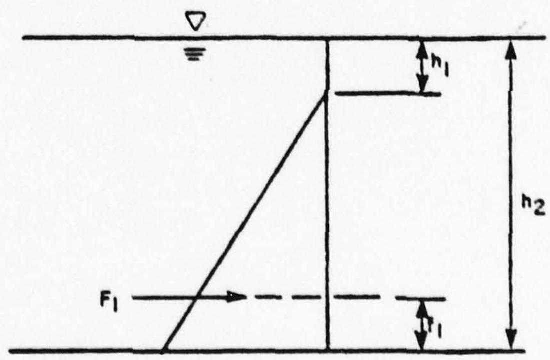
$f_1 = 1/3\ h_2$



Normal Operation

$F_1 = 1/2\ \gamma_w\ (h_2^2 - h_1^2)$

$f_1 = 1/3\ \frac{(h_2 + 2h_1)(h_2 - h_1)}{(h_2 + h_1)}$



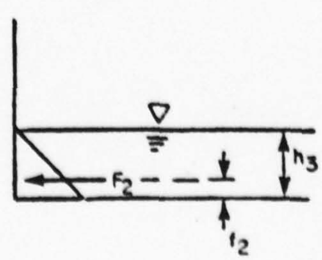
Flood Condition

Tailwater Loads[3]

(overflow section and non-overflow section)

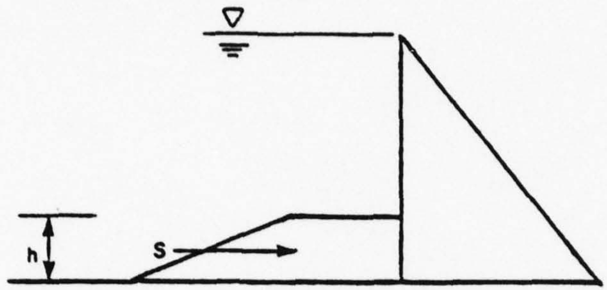
$F_2 = 1/2\ \gamma_w\ h_3^2$

$f_1 = 1/3\ h_3$



Silt Loads

Assume $h = 10'$
Assume 15 $\phi = 30^{o}$



$$S = \frac{\gamma h^2}{2} \frac{(1-\sin \phi)}{(1+\sin \phi)}$$

$s = h/3$

γ_w = unit weight of water

γ = submerged unit weight of silt

γ_d = dry unit weight of silt

γ_{sat} = saturated unit weight of silt

Assume[3] $\gamma_d = 100$ lbs/ft^3

$G_s = 2.6$ (specific gravity)

$n = 40\%$ (void ratio)

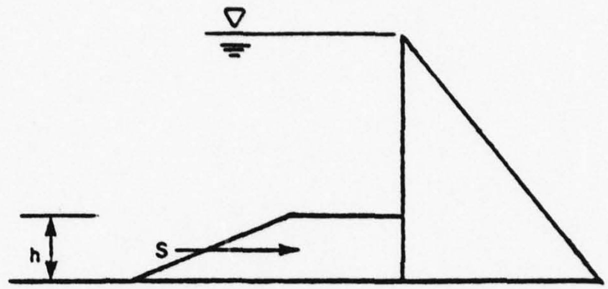
$\gamma_d = G_s (\gamma_w)$

$\gamma_s = (1-n)(\gamma_d) + n(\gamma_w)$

$\gamma = \gamma_s - \gamma_w$

Silt Loads

Assume h = 10'
Assume 15 $\emptyset = 30^{o}$



$$S = \frac{\gamma h^2}{2} \frac{(1-\sin \emptyset)}{(1+\sin \emptyset)}$$

s = h/3

γ_w = unit weight of water

γ = submerged unit weight of silt

γ_d = dry unit weight of silt

γ_{sat} = saturated unit weight of silt

Assume[3] γ_d = 100 lbs/ft^3

G_s = 2.6 (specific gravity)

n = 40% (void ratio)

$\gamma_d = G_s (\gamma_w)$

$\gamma_s = (1-n)(\gamma_d) + n(\gamma_w)$

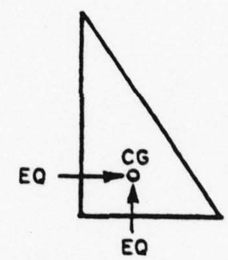
$\gamma = \gamma_s - \gamma_w$

Earthquake Loads[6]

$EQ = \alpha W$

$\alpha = 0.06g$

(acting through center of gravity)



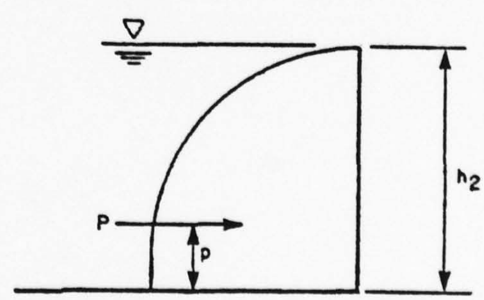
Water Pressure due to Seismic Load [3]

$P = 2/3\ C\ \alpha\ h_2^{\ 2}$

$p = 2/5 h_2$

$\alpha = 0.06g$

$C = 52$



For overflow section, use 95% [3] of water pressure, P.

Penstock Loads [3]
(intake section)

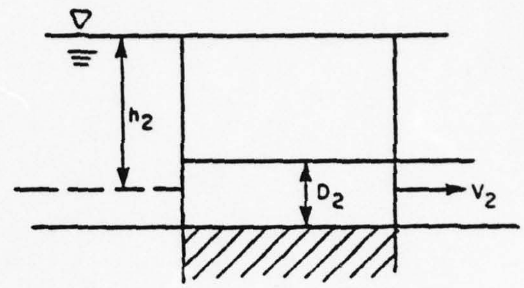
$V_2 = \sqrt{2g\ h_1}$

$A_2 = \pi/4\ D_2^{\ 2}$

$Q = V_2 A_2$

$F_2 = P_w Q\ (V_2)$

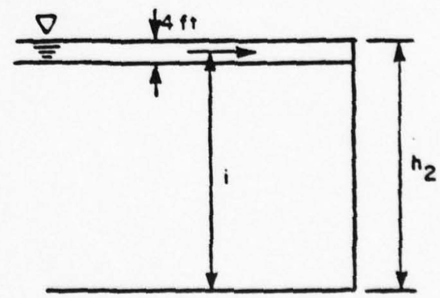
$f_2 = D_2/2$



g = acceleration due to gravity
P_w = mass density of water

Ice Loads [3]
Assume 4 foot thickness

$i = h_2 - 4/2$

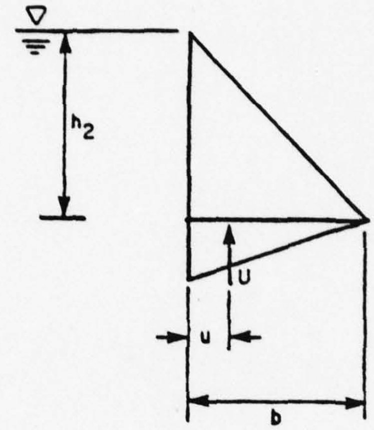


<u>Full Uplift</u> - no tailwater

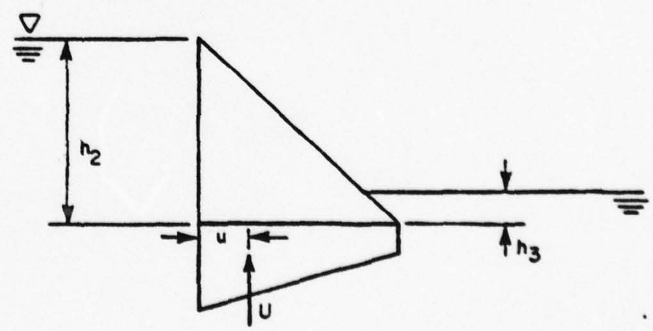
assume linear distribution

$U = 1/2\ \gamma_w h_2 b$

$u = b/3$



- tailwater included

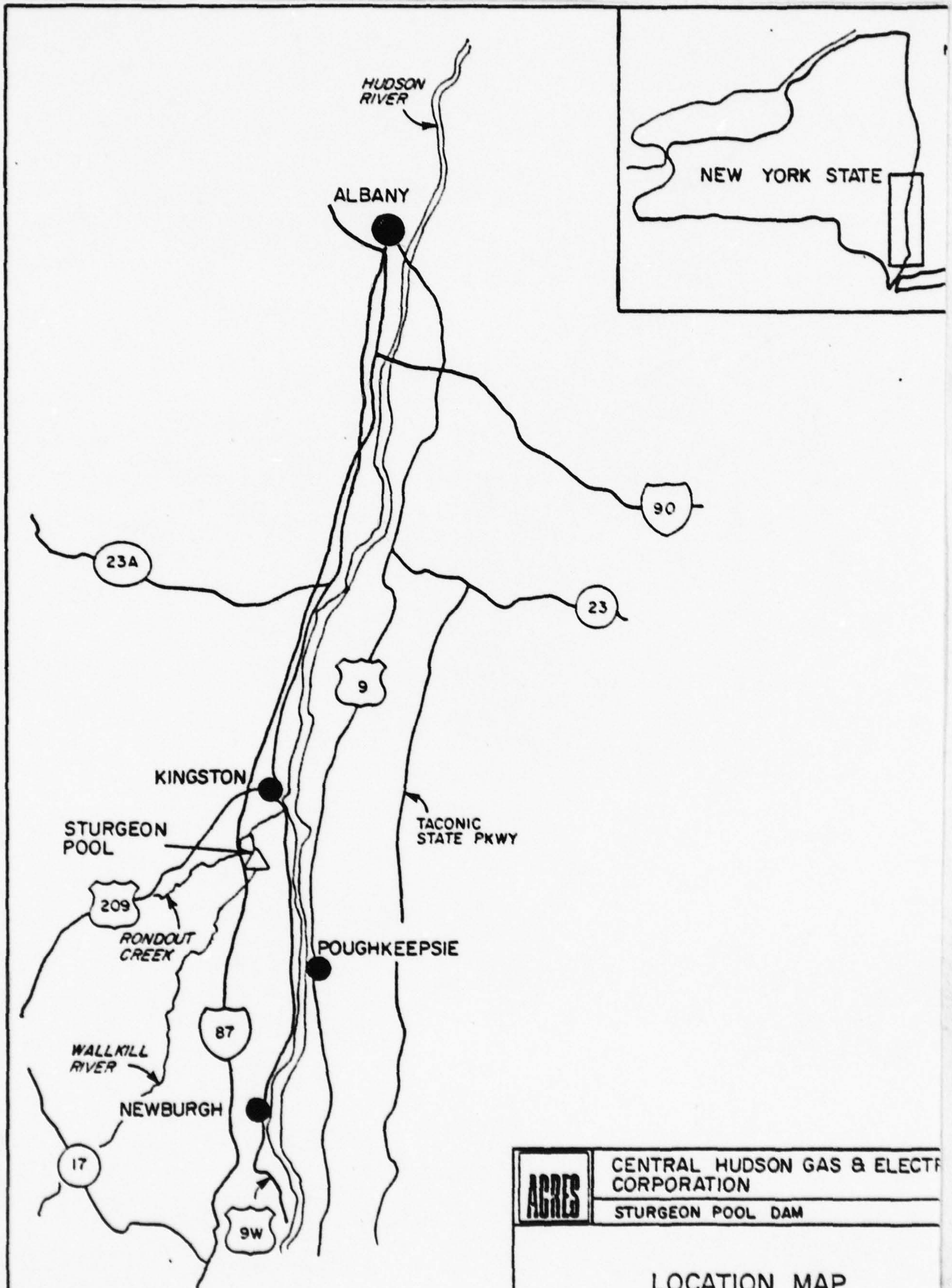


$U = \gamma_w{}^3 b \left[h_3 + (h_2 - h_3)1/2\right]$

$u = h_3 \dfrac{(b/2 + (h_2 - h_3)(1/2)b/3}{h_3 + (h_2 - h_3)1/2}$

UPLIFT - Drains Included

A linear distribution was assumed between headwater and drains located in the lower gallery. The[3] drains were assumed to be partially effective. A head equal to one-third of the difference between the headwater and tailwater was assigned at the drains. A linear distribution between the drain head and the tailwater head was also assumed.



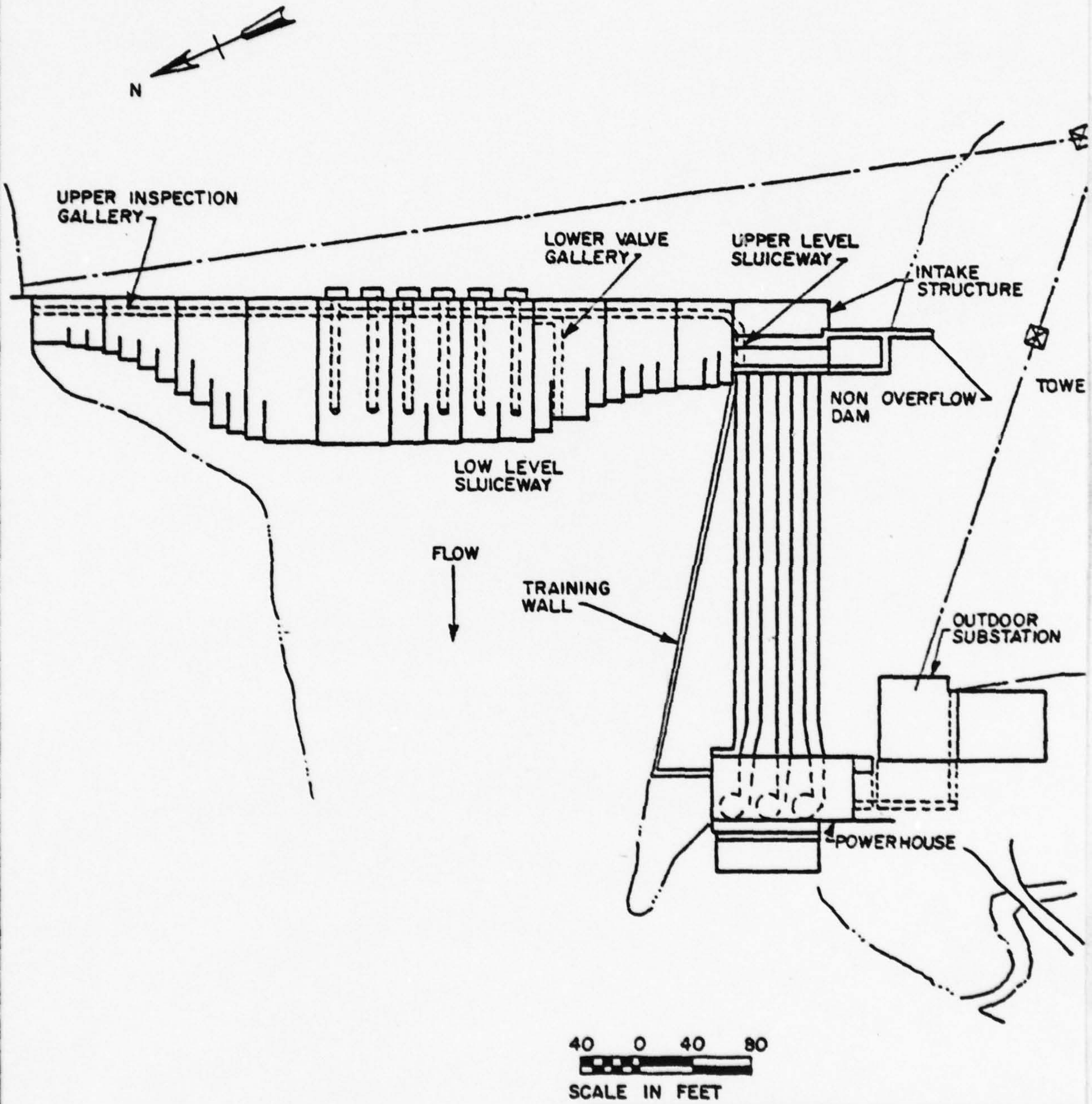
CENTRAL HUDSON GAS & ELECTR
CORPORATION

STURGEON POOL DAM

LOCATION MAP

MAY 1978

PL



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CENTRAL HUDSON GAS & ELECTR
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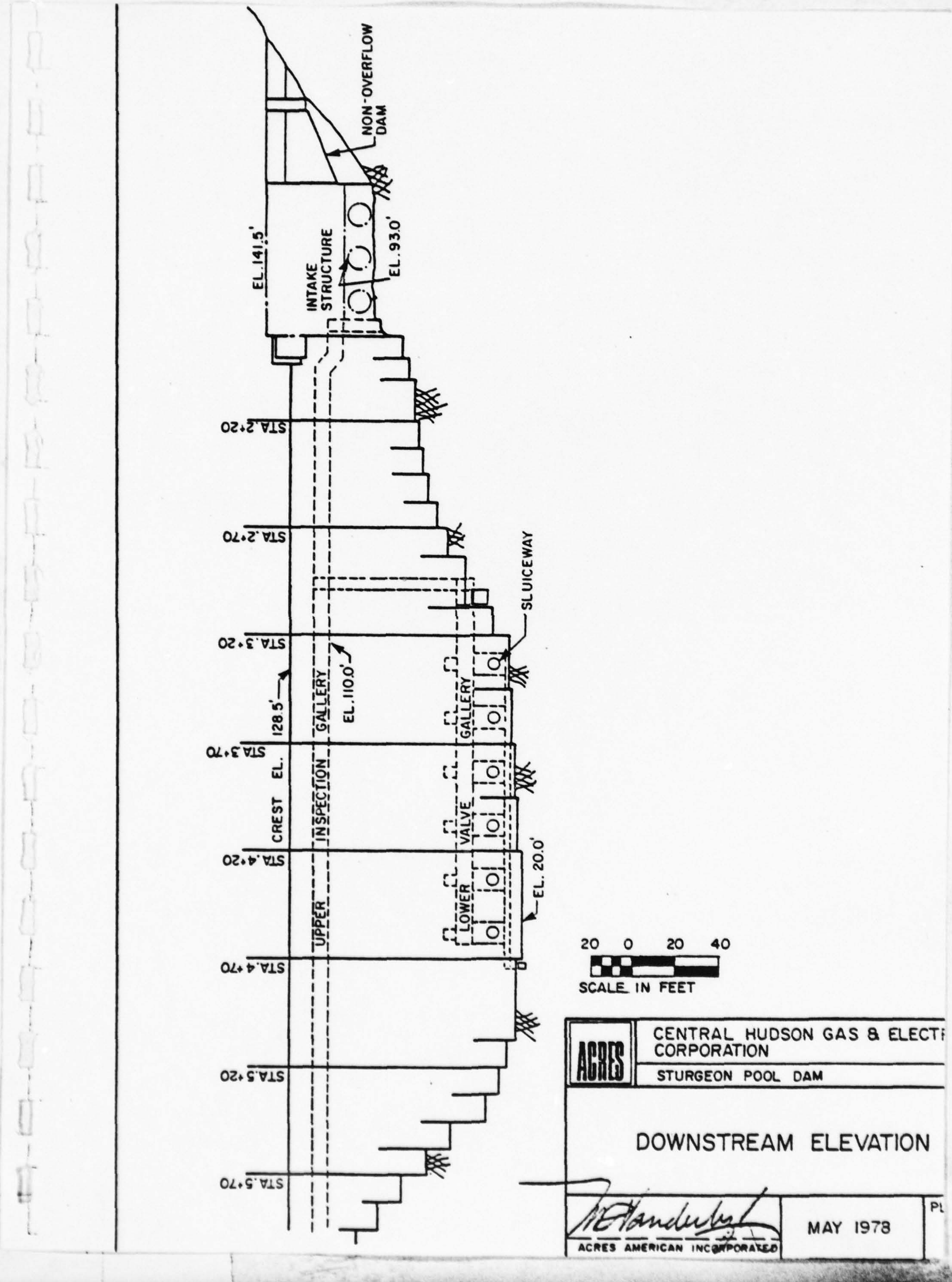
STURGEON POOL DAM

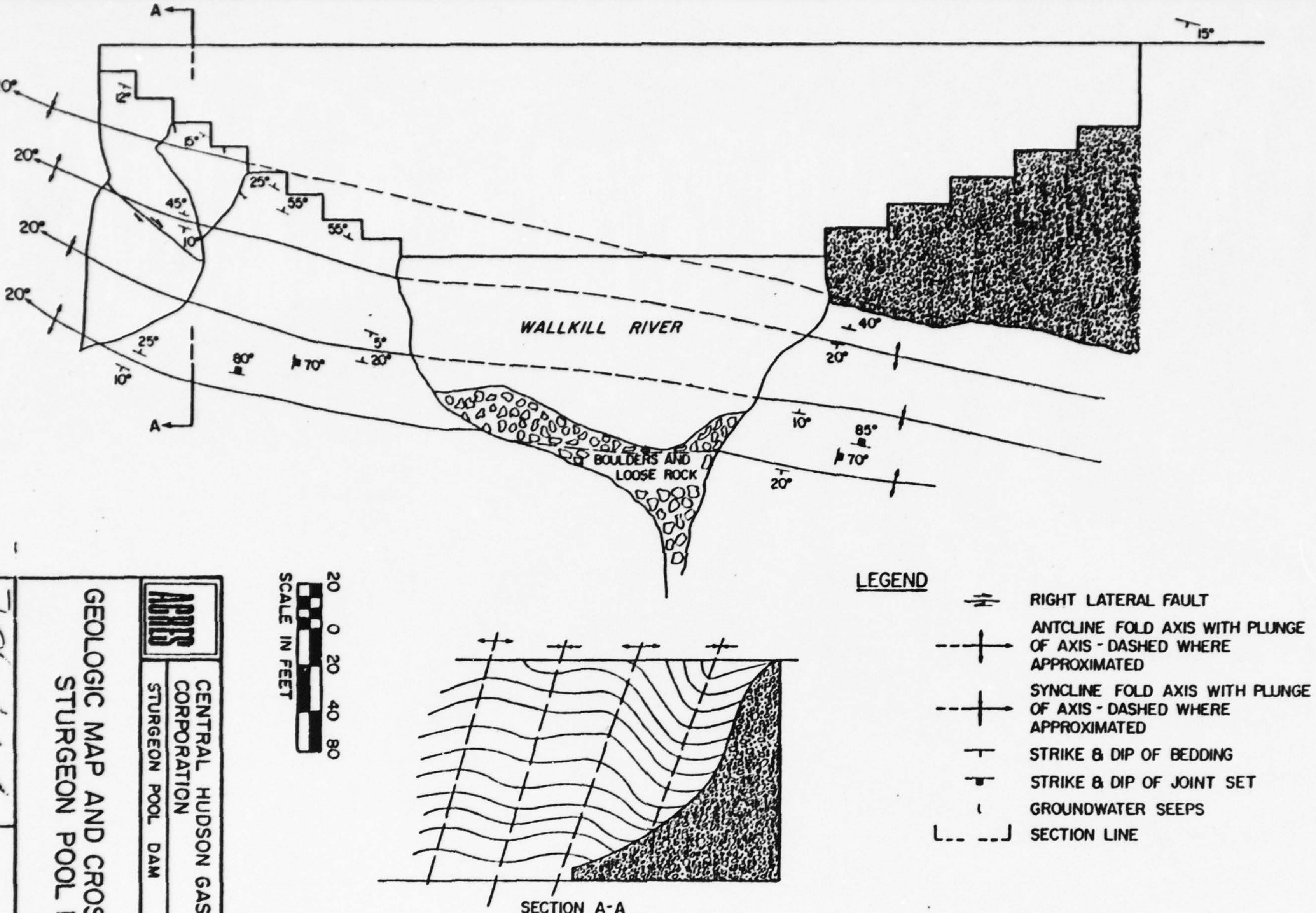
PLAN VIEW
STURGEON POOL DAM

MAY 1978

PL

ACRES AMERICAN INCORPORATED


NON-OVERFLOW DAM
EL. 141.5'
INTAKE STRUCTURE
EL. 93.0'
STA. 2+20
STA. 2+70
SLUICEWAY
STA. 3+20
CREST EL. 128.5'
UPPER INSPECTION GALLERY
EL. 110.0'
STA. 3+70
LOWER VALVE GALLERY
STA. 4+20
EL. 20.0'
STA. 4+70
STA. 5+20
STA. 5+70
20 0 20 40
SCALE IN FEET
ACRES
CENTRAL HUDSON GAS & ELECT
CORPORATION
STURGEON POOL DAM
DOWNSTREAM ELEVATION
ACRES AMERICAN INCORPORATED
MAY 1978



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CORPORATION

STURGEON POOL DAM

GEOLOGIC MAP AND CROSS SECTIC
STURGEON POOL DAM

ACRES AMERICAN INCORPORATED

MAY 1978

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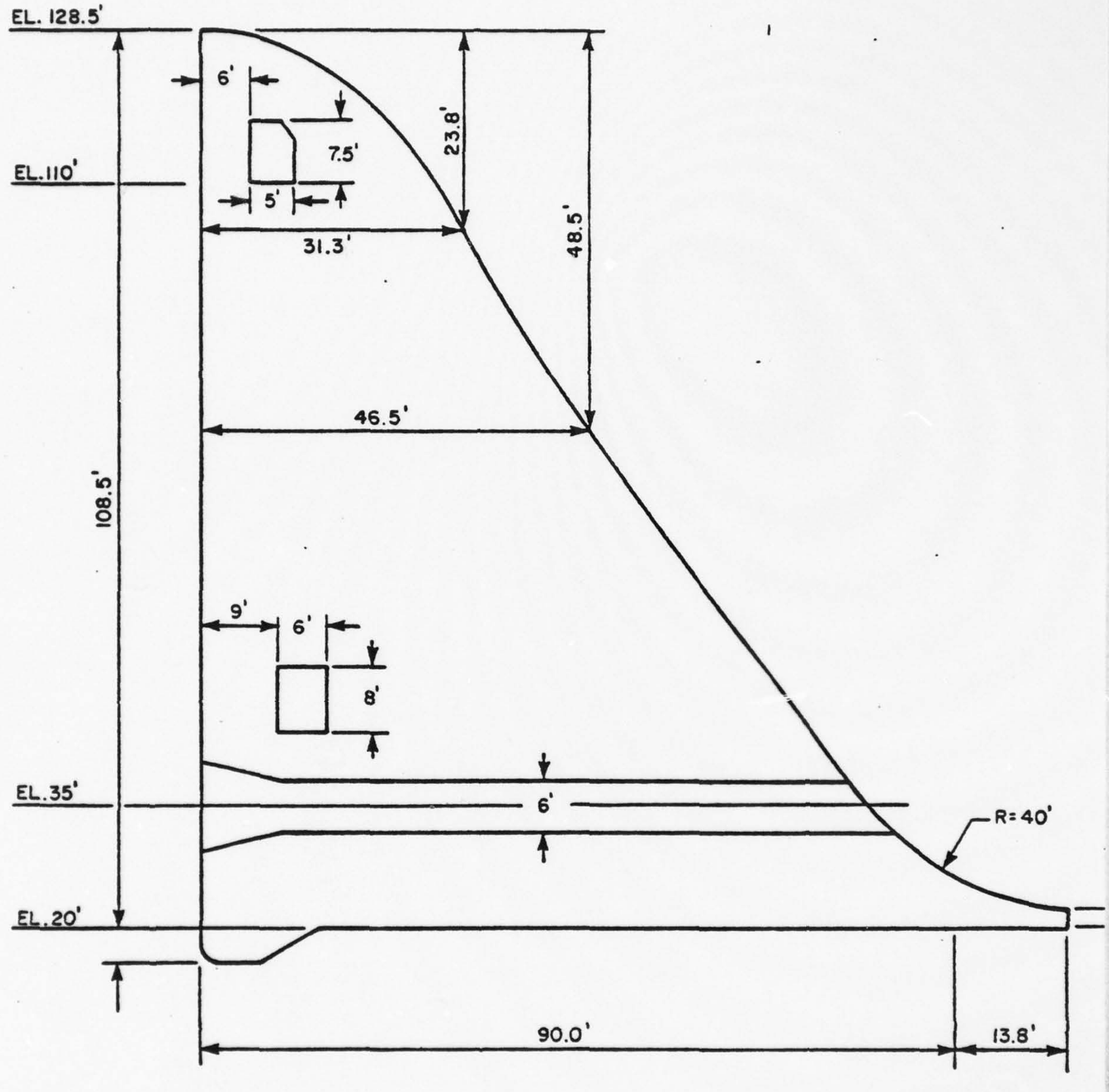

SCALE = 1" = 20'

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CORPORATION

STURGEON POOL DAM

OVERFLOW CROSS SECTION

ACRES AMERICAN INCORPORATED

MAY 1978

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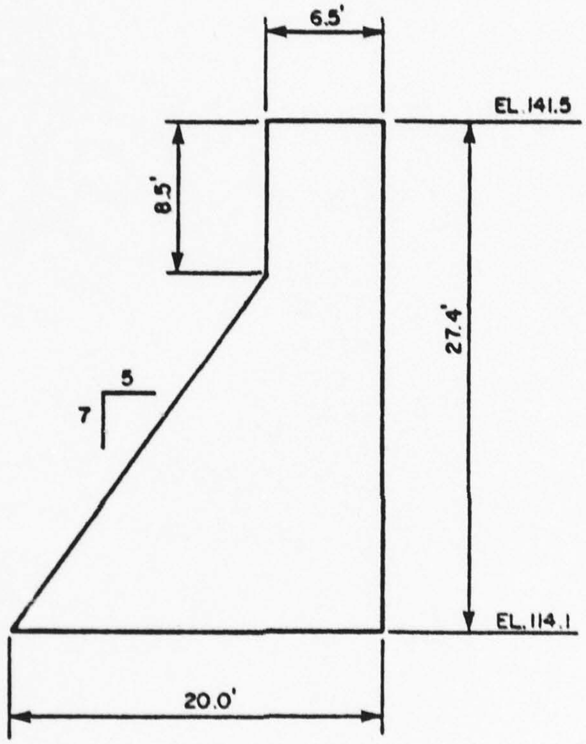

SCALE = 1" = 10'

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STURGEON POOL DAM

NON-OVERFLOW SECTION

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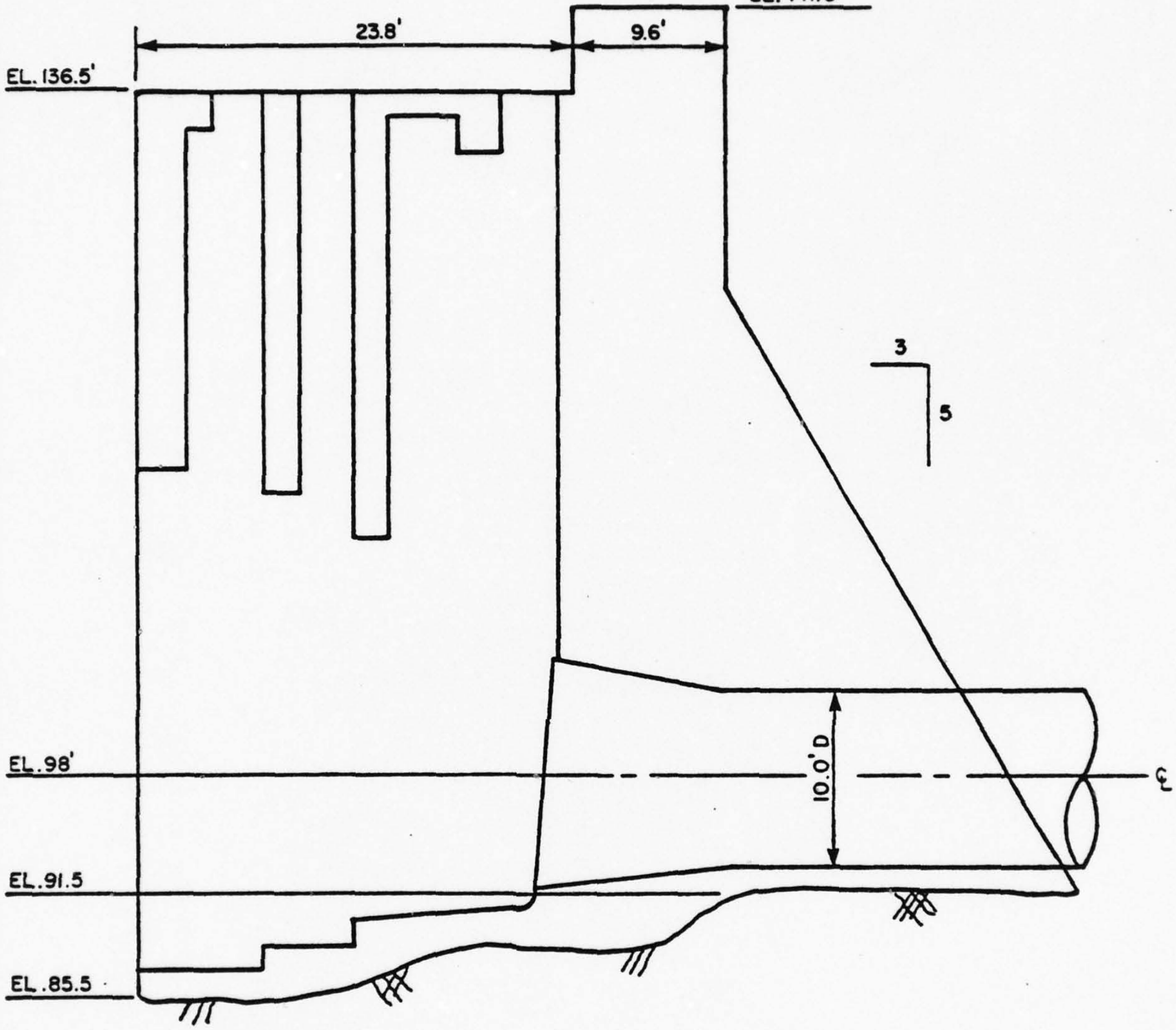

SCALE = 1" = 10'

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CORPORATION

STURGEON POOL DAM

SECTION THROUGH CENTERLI
OF PENSTOCK INTAKE

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MAY 1978

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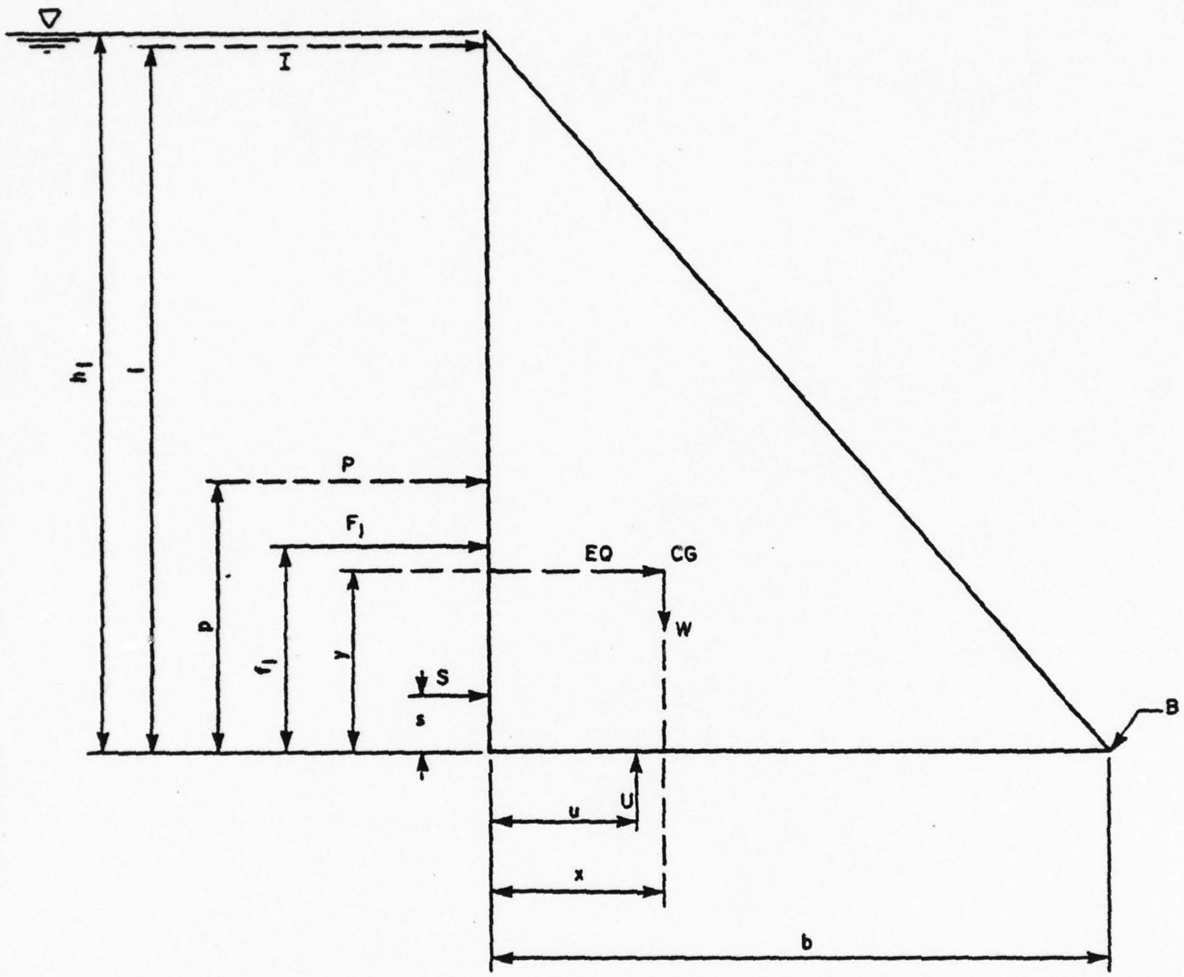

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STURGEON POOL DAM

LOADING CASE UNDER NORMA
CONDITIONS STATIC AND
SEISMIC LOADS

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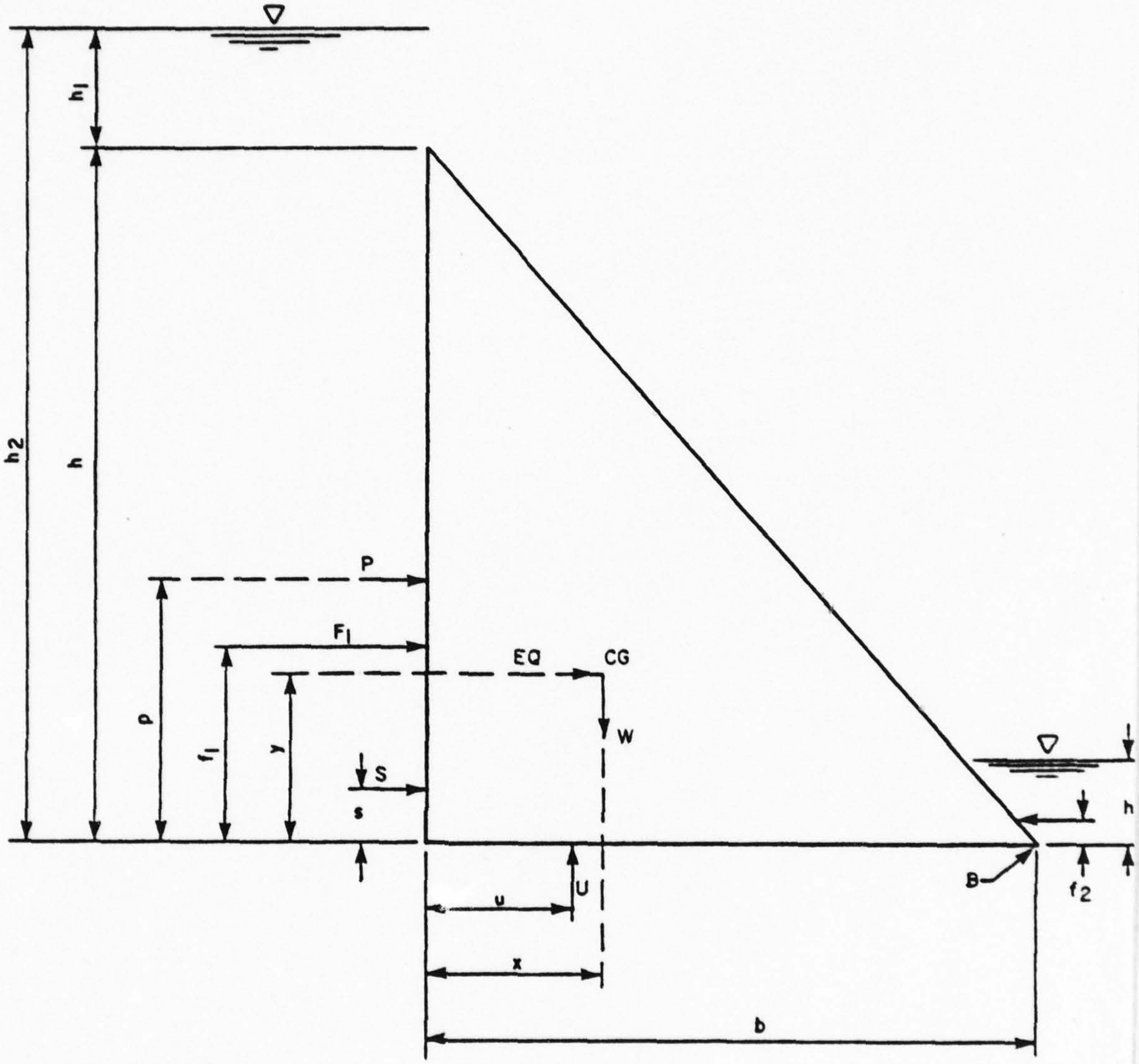

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CENTRAL HUDSON GAS & ELECT
CORPORATION

STURGEON POOL DAM

LOADING CASE UNDER
FLOOD CONDITIONS
STATIC AND SEISMIC LOADS

ACRES AMERICAN INCORPORATED

MAY 1978

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3 Creager, William P. and Joel D. Justin. Hydroelectric Handbook. John Wiley B. Sons, Inc.; New York, 1950.

4 Deere, Don U. "Dams on Rock Foundations - Some Design Questions". Rock Engineering for Foundations & Slopes. American Society of Civil Engineers; New York, 1976.

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16 U.S. Department of the Interior, Concrete Manual, United States Government Printing Office; Washington, 1975.

17 U.S. Department of the Interior, Design of Small Dams, United States Government Printing Office; Washington, 1974.

APPENDIX A

DRAFT

CENTRAL HUDSON GAS
AND
ELECTRIC CORPORATION
STURGEON POOL DEVELOPMENT

REPORT ON
RESTORATION & REPAIR
OF STURGEON POOL DAM

MAIN
CHAS. T. MAIN OF NEW YORK, INC.

June 1978

1050-35

DRAFT

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DRAFT

INTRODUCTION

The present report summarizes the findings of MAIN's most recent investigation into the current condition of the dam, intake structure and penstocks of the Sturgeon Pool Development. This report should be read in conjunction with the test report issued by the Thompson and Lichtner laboratory in March, 1978, entitled, "Report of Coring Operations and Concrete Evaluation". Additional reference material is contained in three previous inspection reports for the same project issued by Chas. T. MAIN under the following dates: December 1973, September 1973 and December 1970.

Because of the extensive deterioration noted during the inspection and because of the client's urgent concern, this report also contains specific recommendations for restoring the structures to their original condition, including estimates of the associated costs. An appendix to the report contains drawings outlining the repair work to be done and a copy of the environmental statement that is required for a repair permit from the D.E.C. for New York.

FIELD INSPECTION & SURVEY

Sturgeon Pool Dam is a concrete gravity structure scaling about 108 feet in height with an ogee type spillway measuring 490 feet in length between abutments. The right abutment is a non-overflow bulkhead. The left abutment is comprised of a high level discharge gate adjoining a concrete intake structure, and terminating in a non-overflow bulkhead. Shortly after construction the downstream face of the dam was paved with granite blocks varying in thickness from 8 inches at the crest to 16 inches at the toe. At 50-foot intervals, as measured parallel to the crest, the dam was constructed with vertical contraction joints extending from crest to base, which in effect divide the basic structure of the dam into 10 discreet concrete blocks. During the dry season 4-foot high flashboards are strung along the crest of the dam. (These boards are held to 3 feet in the center third of the crest.) Calculations performed during earlier inspections show the structure to be stable against overturning or sliding under all assumed loading conditions. The Field inspecti[on &] survey did not reveal any structural damage that would chang[e] stabi[lity] of t[he] stab[ility]. The current investigation was initiated by an alarming deterioration ~~increase in the rate of destruction~~ of the granite facing on the dam, combined with a steady increase in the infiltration of water into the inspection galleries. When the facing was originally installed over the concrete substructure, it was backed up by an extensive system of drains intended to collect seepage through contraction joints and deliver the water to outlets at the downstream toe of the dam. At the present time, this facing is showing extensive surface damage as a result of the freezing and thawing action of water which has seeped behind the blocks. In several areas blocks are missing entirely, while at scattered other locations a general outward bowing of the granite facing is evident. At the same time, an undetermined depth of underlying concrete, particularly at the crest, has softened and deteriorated from the same freeze-thaw cycle.

Over a period of years, many of the metal seals provided at the contraction joints have failed, permitting water to flow into the upper inspection gallery. ~~to a depth of 6-inches or more~~. Metal pans have subsequently been installed at the most affected joints to prevent water from spraying completely across the gallery. Most of this water is channeled into the previously mentioned system of drains. As a consequence, this system, which originally was adequate to handle a normal amount of seepage, is now overloaded, and is actively contributing to destruction of the face of the dam by conducting gradually increasing amounts of water to previously undamaged areas.

Water tight integrity of the granite facing has been further jeopardized by a detail which calls for every course of block to pass directly across a contraction joint in the underlying concrete. Since the annual variation in temperature of the underlying concrete is not less than 50° F (the variation in ambient air temperature exceeding 100° F), this implies an annual movement across the joint of not less than 1/8 inch, A movement of this magnitude is sufficient to loosen the granite block from its mortar embedment. At the crest, water tightness has been additionally compromised by the presence of two embedded crane rails, which served at one time to support a traveling crane for opening the now-abandoned low level discharge gates. Where the crane rail crosses the contraction joints, differential movement between the rail and block has broken the mortar joints fastening the block to the dam.

The lower inspection gallery was free of any flowing water at the time of inspection. Surfaces were damp, perhaps partially due to condensation build-up, and generally calcified on the ceiling and upstream wall portions. The six sluiceway valves are accessable from this passageway. Most of the valves are said to be mechanically inoperable at this time. Even if they could be opened, the amount of silt then discharged downstream would not be acceptable. Except for a 60 gpm flow through one of the 6-foot diameter sluiceways, all valves were functioning effectively to seal off the upper pond.

The upstream face of the dam was inspected during a 24-foot drawdown of headwater. Many of the horizontal construction joints were visible because of poor concrete placing techniques during construction, particularly by the presence of segregation. An effort to seal all vertical contraction joints had been attempted at some time in the past by covering the joints with a gunnite -on-wire-mesh batten. All of these battens have subsequently failed in their intended purpose.

The situation is best described in the following quotes from a letter by Central Hudson personnel accompanying the inspection,

> "With use of a boat at the 105 foot level, it was noted that horizontal pour joints at the 110 level showed extensive erosion between the fourth and fifth vertical construction joint from the north end. The depth of opening at the joint varied from about four to ten inches. Some silt was noted in the joint which

could indicate that some seepage does occur through the joint. The extent of seepage is not visible on the downstream face of the dam."

When the pond level reached 120 feet, Central Hudson personnel

"...conducted an inspection of the upper level of the dam. Extensive deterioration of the grout between the granite blocks was noted for the southern four hundred feet of the dam. The area extended the length of tracks on the dam for the crane and from the crest to about five feet down on the wet face. Gaps were noted between the granite blocks and the grout. In some areas the grout could be broken and removed. The worst conditions were in the areas that the downstream granite blocks lifted from the face."

It should be noted that conditions observed in the second quote will be corrected by the repair work scheduled for the crest this summer.

Located at the left abutment of the ogee is a concrete intake structure with three crane-operated intake gates controlling the flow of water to three 10-foot diameter steel penstocks. Except for the upstream face, this structure is also faced with granite blocks. The roof of the structure is an outdoor deck paved with hard-burned brick. ~~or tile.~~ Between the intake structure and the crest of the dam is an upper level sluiceway. The crane-operated gate installed in this sluiceway controls an opening 10 feet wide with a sill 8.6 feet below the dam crest.

When the water level of the pond was drawn down 24 feet below the dam crest, the upstream concrete face of the intake structure was exposed to reveal a generally satisfactory condition. There was a narrow band of concrete just above the intake openings where flow pattern had eroded the surface sufficiently to expose the aggregate. Two vertical contraction joints, as well as a number of horizontal construction joints were visible. It was not possible under existing conditions to tell which, if any, of these joints under full pond conditions might be conducting water to the underside of the penstocks.

The brick paved deck surmounting the intake structure has been damaged and loosened by the same freeze-thaw action which has served to loosen the granite paving on the dam. As a consequence several areas of the granite masonry covering the intake structure below deck level show evidence of water seeping behind the stones from above. At several locations the personnel guard rail surrounding the deck is no longer securely attached to the brick paving.

relocate paragraph

At the present time a small flow of water is occurring under both penstocks No. 1 and No. 2 at the point where the penstocks

exit from the concrete of the intake structure. While the steel pipe portion of the penstock shows pitting from rust at both these locations, the pipe itself does not appear to be porous. Rather there are indications that the leakage is occurring through concrete joints in the intake structure itself. It is significant that investigation by Central Hudson personnel showed no flow occurring under either penstock when headwater had been drawn down about 24 feet to El. 105, and that flow did not resume under the middle penstock until headwater had been restored to El. 123. This situation will be investigated more fully during the summer when the pond elevation has been lowered for construction. Pressure grouting of concrete joints is indicated as the method of treatment here.

CONCRETE CORING TEST REPORT

During the month of March, 1978, Thompson and Lichtner laboratory was instructed to remove concrete cores from the crest of the dam for compression testing and chemical analysis, and to sample sedimentary deposits bearing on the upstream face of the dam. Subsequent grading tests performed on this sedimentary material showed it to be composed of fine sands, medium sands and well graded sands with some silt.

Thirteen concrete cores were removed by boring upward from the interior of the gallery to the underside of the granite blocks at the crest. The cores were taken at more or less equal spacing along the gallery and at one of four different angles from the vertical. Actual locations and angles are as shown in Drawing 1050-35-SK1 included in the Appendix.

Compression tests of the cores indicated a wide variation in concrete strength, ranging from a low of 1180 psi to a high of 4290 psi. However, out of 25 sample compression tests performed 15 cylinders failed at pressures above 3000 psi and 6 additional cylinders failed above 2300 psi. Two of the remaining 4 cylinders failded due to subsurface deterioration of the concrete from the previously mentioned freeze-thaw cycle. An additional cylinder failed because of the presence of a 5-inch diameter piece of coarse aggregate in a 6-inch diameter core.

Chemical analyses of 8 concrete cores indicated cement contents ranging from 128 pounds of cement to 468 pounds of cement per cubic yard, with 6 of the cores having cement contents of 380 pounds or above. Grading of the coarse sizes (plus #4 sizes) of aggregate indicated a generally coarse grading, while grading of the fine sizes (sub #4 sizes) indicated the sands to be very fine. A modern concrete mix would require a much more uniform distribution both between and within the coarse and fine aggregates. Because of the wide variations in strength, cement content and size of aggregate, the concrete mix as placed in the dam is far from what would be called a "controlled" mix today.

DRAFT

REPAIRS & ALTERATIONS - 1978 STAGE

Repairs and alterations as presently proposed conveniently divide into three schedules. Immediate repairs are those which must be undertaken during the summer of 1978 to restore the watertightness of the dam and prevent further loss of granite facing during the following winter. Intermediate stage repairs to end seepage around penstocks at the intake structure and to secure the low-level outlet with permanent concrete plugs can be undertaken during the 1979 construction season. Long-term repairs can be undertaken as routine items according to the accompanying repair schedule or as unusual conditions arise between inspections.

Immediate (1978) repairs are listed below. Their estimated cost as totaled in Table I amounts to $680,000 (if completed in two construction seasons), or to $968,000 if completed in one construction season.

1. The entire granite and concrete crest of the dam, including the embedded crane rails, will be removed to a depth of not less than 3-feet and a width of 16-feet, and will be replaced by a new concrete cap of approximately the same dimensions, reinforced with temperature steel. The contraction joints occurring at 50-feet intervals will be furnished with a dumbell-type rubber waterstop.

Correct entire para

2. The existing vertical contraction joints will be drilled from crest to ~~base~~ elevation 180.0 with ~~four~~ one 4" diameter ~~holes at 3½" on center, forming a slot~~ hole. A dumbell-type rubber waterstop will be inserted into the slot for its entire height, and will be held in position by non-shrink grout poured into the remaining voids. When pool is drawn down, horizontal construction joints exposed in upstream face will be patched and grouted where necessary.

Not in 78

3. All granite blocks on the downstream face of the dam, presently crossing or abutting the vertical contraction joints, will be removed and replaced with new concrete dental work of the same dimensions, but with the contraction joint now carried through to the surface.

Not in 78

4. At locations on the downstream face of the dam, other than at the contraction joints, where granite blocks have loosened or fallen out, blocks will be removed, cleaned and reset in new mortar, utilizing the present system of metal bar anchors where possible.

Not in 78

5. Instead of being totally interconnected as at present, the existing network of face drains will be separated into discreet horizontal units of not more than 50-feet in length, and each unit will empty itself through an outlet at the face of the dam near the vertical contraction joint.

REPAIRS & ALTERATIONS - 1979 STAGE

Intermediate (1979) stage repairs are listed below. Their estimated cost as totaled in Table II amounts to $130,600.

6. Remove all electric motors and accompanying wiring and steel flooring from the low-level sluiceway valve chambers. Place permanent concrete plugs in the sluiceways and valve chambers as illustrated in MAIN's December 1973 report, Plate III.

7. Remove existing brick paved slabs on upper and lower outdoor decks of the intake structure and replace with 8-inch air-entrained concrete slabs with water repellent surfaces.

Add
work removed
from 78

8. Reset or replace existing personnel railings around perimeter of new deck slab on top of intake structure. As part of this operation it is recommended that a new chain link fence topped with 3 strands of barbed wire be fitted to the top of the left non-overflow bulkhead.

9. Pressure grout any vertical and/or horizontal construction joints in the intake structure which appear to be conducting water to the undersides of the steel penstocks as the penstocks emerge from the concrete monolith.

10. Patch externally rusted areas of steel penstocks with steel plate at invert where penstocks emerge from intake structure monolith.

ROUTINE REPAIR SCHEDULE

The routine repair schedule is largely abstracted from the recommendations made in MAIN's September 1973 report on the subject project. The time of repair is given to help schedule the work systematically and to indicate the relative urgency of the repair. "As required" items are not scheduled for any definite time, but should be observed in order that the most appropriate time for their being attended to should be chosen.

REPAIR SCHEDULE

	Location	Type of Repair	Time of Repair
1.	Left Non-Overflow Bulkhead	Grout abutment to stop seepage	Within 10 years o as required
2.	Intake Structure	Rake and point mortar joints in granite block facing	Within 5 years or as required
3.	Penstocks	Seal leaks in penstocks	Within 5 years or as required
4.	Spillway	Paint or replace spiral staircase	As required
5.	Spillway	Construct protective concrete walls and slabs over eroding rock downstream of spillway	As required
6.	Right Non-Overflow Bulkhead	Repair stairway and soffit	Within 5 years
7.	Right Non-Overflow Bulkhead	Provide weatherproof doors with louvers and repair deteriorated concrete	Within 5 years or as required
8.	Downstream of Spillway Toe	Monitor eroding rock and protective concrete slabs and walls	Annually

TABLE I
COST SUMMARY
FOR
IMMEDIATE (1978) REPAIRS
STURGEON POOL DEVELOPMENT

Item No.		Work or Material	Cost
1		Concrete Replacement Cap	626,700
2		Drilling & Resealing of Vertical Contraction Joints	52,500
3	re-do	Concrete Dental Work at Contraction Joints on Downstream Face	112,600
4		Resetting of Displaced Granite Block on Downstream Face	11,700
5		Rerouting of Face Drains	3,000
		Subtotal	806,500
		Contingencies (20%)	161,300
		Total Cost of Repairs	967,800

TABLE II
COST SUMMARY
FOR
INTERMEDIATE (1979) REPAIRS
STURGEON POOL DEVELOPMENT

Item No.	Work or Material	Cost
6	Concrete Plugs in Low-Level Sluiceways	93,000
7	Concrete Slab on Intake Structure Deck	8,100
8	re-do Handrailing Around Intake Structure Deck	2,700
9	Pressure Grouting, Intake Structure (Including Drilling)	2,500
10	Patching Steel Penstocks at Intake	2,500
	Subtotal	108,800
	Contingencies (20%)	21,800
	Total Cost of Repairs	130,600

APPENDIX A

ENVIRONMENTAL OVERVIEW
STURGEON POOL DAM REPAIRS

Summary of Proposed Action

Sturgeon Pool Dam and the associated powerhouse were completed in 1923. Shortly thereafter, the concrete masonry structure was faced with grouted granite block on the downstream side of the ogee spillway. Recently seepage and surface water, in combination with freezing/thawing cycles, has displaced sections of block immediately below the crest of the spillway. Although public safety is not jeopardized by this condition, it is Applicant's intention to correct the problem before it becomes more serious. The proposed remedial program is summarized as follows:

1. The removal of approximately six (6) vertical feet of block and concrete from the crest of the existing spillway.
2. Reconstruction of the crest section with reinforced concrete keyed mechanically to the existing structure.
3. Sealing of vertical construction joints by drilling and pouring sealant.
4. Remedial repairs to the facing drainage system.

The proposed action is described in further detail in the technical section of this report.

Existing Environment

Sturgeon Pool is a 210-acre impoundment with a maximum depth of 108 feet. The dam is located approximately one-half mile above the confluence of Rondout Creek and Wallkill River, which is the primary source of supply to the reservoir. The Wallkill River and Rondout Creek, both above and below the confluence of the two, are denoted Class B and Standard B in Chapter X of the "Conservation Law". *

* "State of New York, Official Complation of Codes, Rules and Regulations", Title 6, Department of Environmental Conservation.

Although the reservoir's primary function is to provide storage and head for power generation, access for recreational use has historically been afforded to residents living adjacent to Applicant's on-shore property holdings. Recreational facilities at Applicant's Rifton Recreational Center (northwest end of the reservoir) are maintained for use by employees and their guests. Boating, swimming, fishing and waterskiing are identified as the primary recreationa uses. With respect to swimming, public health service testing has indicated that water quality is adequate for such usage except for occasional occurrences of excessive algae bloom. Although quantitative data is not available, an examination of color aerial photography identifies the Wallkill River as the primary source of turbidity in the reservoir. There are no large areas of contiguous wetland to Sturgeon Pool. Reservoir slopes are characterized as stabilized and vary from gently sloping on the east bank to moderately steep on the west bank. No fisheries management program is in effect and fishing is generally considered to be poor due to the abundance of carp and the absence of abundant populations of desirable species (per discussions with Rifton Recreational Center personnel).

Probable Environmental Impacts

The potential impacts of the proposed action are summaried below. It is important to note that the impacts are temporary (Summer of 1978)* in nature and that similar impacts have been experienced in the past to facilitate maintenance operations. The primary factor contributing to potential impacts is the necessity to draw down the reservoir approximately ten (10) feet and maintain that level during the 1978 Summer construction period.

A. Upstream Impacts

1. Visual - It is estimated, from previous drawdown experience,

* Two summer construction seasons may be required due to the magnitude of the effort.

that a band of reservoir slopes averaging 40 to 50 feet in width (and longer peninsulas) will be exposed. The primary visual impact will be to abuttors and to motorists traveling on Route 213.

2. Sheet Erosion - The exposure of the relatively stable reservoir slopes will slightly increase potential for sheet erosion to the reservoir. The impact is considered insignificant compared to the amount of erosion from the entire watershed area (805 square miles) which is transported to Sturgeon Pool via the Wallkill and other tributaries. (This conclusion is based on in-depth studies for pumped storage power projects.)

3. Wildlife - No rare or endangered species have been identified to date. There are no extensive wetlands adjacent to the reservoir which would be drained temporarily by the proposed action. The drawdown could adversely affect fish propagation, particularly with respect to nest-building fish but this will be minimal, as such fish are expected to rebuild their nests if exposed during the drawdown period. (Spawning season is approximately May to July.) Experience from pumped storage power projects also suggests that fish will continue to propagate even if fluctuation within the drawdown zone equal to the magnitude of drawdown is experienced during the construction period.

4. Recreation - Use of the reservoir for recreation during the 1978 season will be affected in that the employee recreation swimming area will be closed in the interest of safety due to steep underwater slope conditions. Other day use facilities, such as picnicking, playground activities and softball will be unaffected

by the proposed action. Swimming and other recreational use by abuttors may be similarly affected but not necessarily precluded, depending on the conditions at each site.

B. Downstream Impacts

1. River Regime - The remedial repair program is scheduled for the dry Summer period (after Spring run-off). Drawdown to facilitate construction and maintenance of this temporary level will be accomplished by use of the penstocks leading to the power generation facility and thence to the tailrace. This discharge method is consistent with normal summer operating procedures when the facility is used for approximately 4 to 6 hours per day (only) for peaking power due to the limited water supply. A temporary reduction of generating capacity due to reduced head is identified as an additional minor impact. Incremental changes to the river regime, if any, will be mitigated further downstream by normal flow in Rondout Creek at its confluence with the Wallkill..

2. Construction Debris - Debris associated with the removal of the crest and other construction material, such as forms, will be minimized by directing the Contractor in the construction documents to install such measures and employ suitable techniques to preclude the loss of materials to the reservoir or to the Wallkill River. The specifications will require that the Contractor submit a debris control plan which will be subject to the Engineer's approval.

3. Silt Transport - Field investigations have established the presence of silt adjacent to the existing low level outlets

of the structure. Therefore, it is Applicant's intention to maintain the level of the reservoir after drawdown with the exclusive use of the penstocks. Silt transportation will, therefore, be limited to silt normally in suspension which comes primarily from the Wallkill River watershed and in insignificant amount of additional material from the temporarily exposed reservoir slopes.

Direct Impacts of Construction

A. Laydown Area

The Contractor will be permitted use of the relatively small area at the north abutment of the dam for construction purposes which has previously been cleared and graded. Damage to the area, if any, will be repaired by the Contractor at the conclusion of his work.

B. Disposal Area

Approximately 1200 cubic yards of concrete removed from the crest of the dam will be disposed of on Applicant's property approximately one-quarter mile northwest of the north abutment. Access to the site will be via an existing paved road and then via off-road access to the site. The disposal operation will involve clearing approximately one-quarter to one-half an acre of trees, spoiling the material above grade and restoring the site with the addition of organic soil and vegetative cover (grass). The specific site will be selected to minimize erosion.

C. Noise

Construction noise, particularly that associated with drilling and removal of concrete, will be scheduled insofar as practicable not to

disturb local residents who are as close as one-half mile from the site (to the east and to the west).

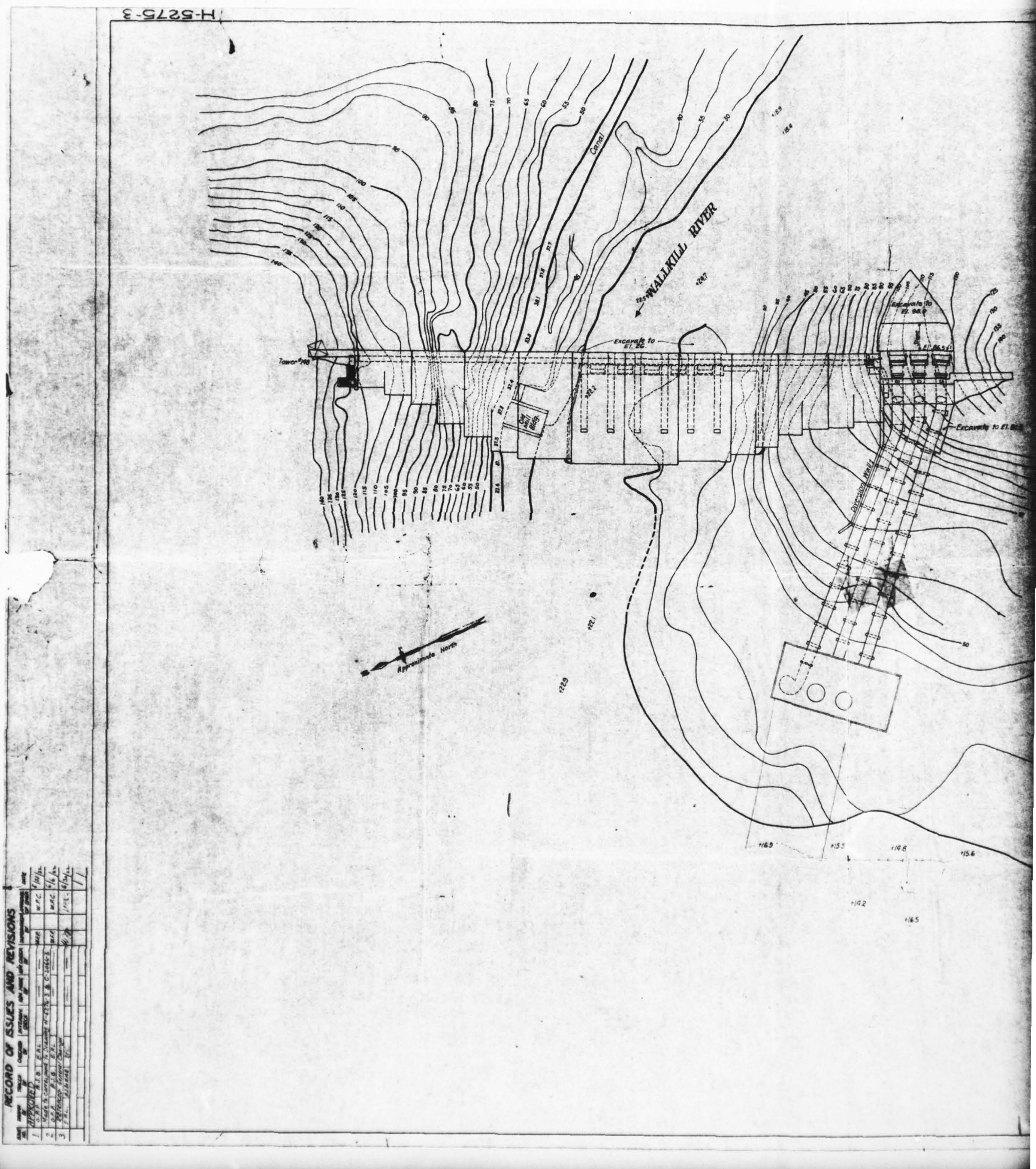
D. Dust

Dust is not expected to be a major impact associated with this action. Earthwork will be limited to the disposal area, the majority of access is paved, and the dust from concrete removal will be minimal considering the economy of removing it in relatively large sections.

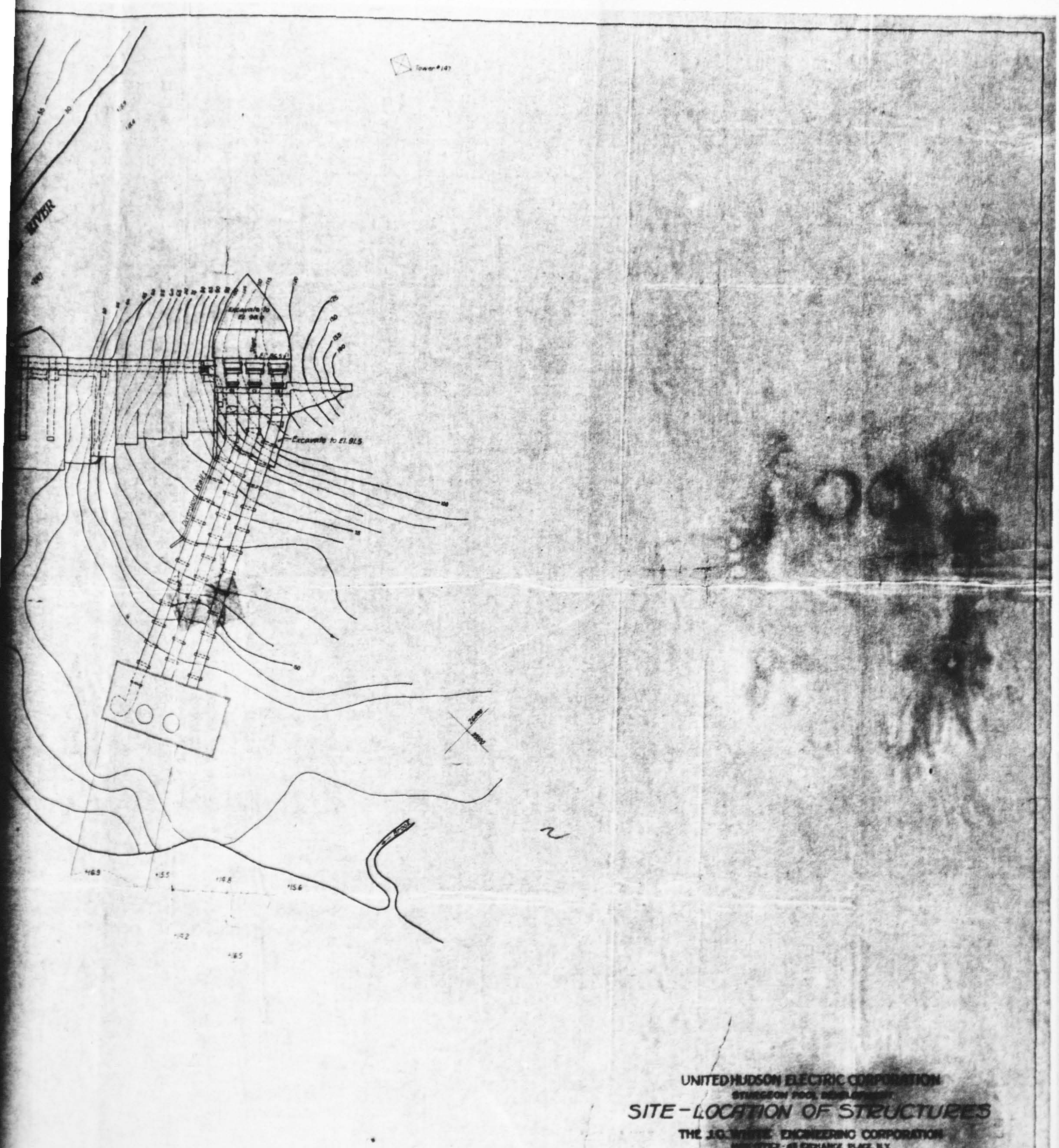
In summary, the identified impacts are temporary in nature and will not require a significant commitment of irretrievable resources. Positive benefits include the repair of the spillway and the positive influence of the purchase of material and labor which may have a beneficial effect on the local economy.

APPENDIX E

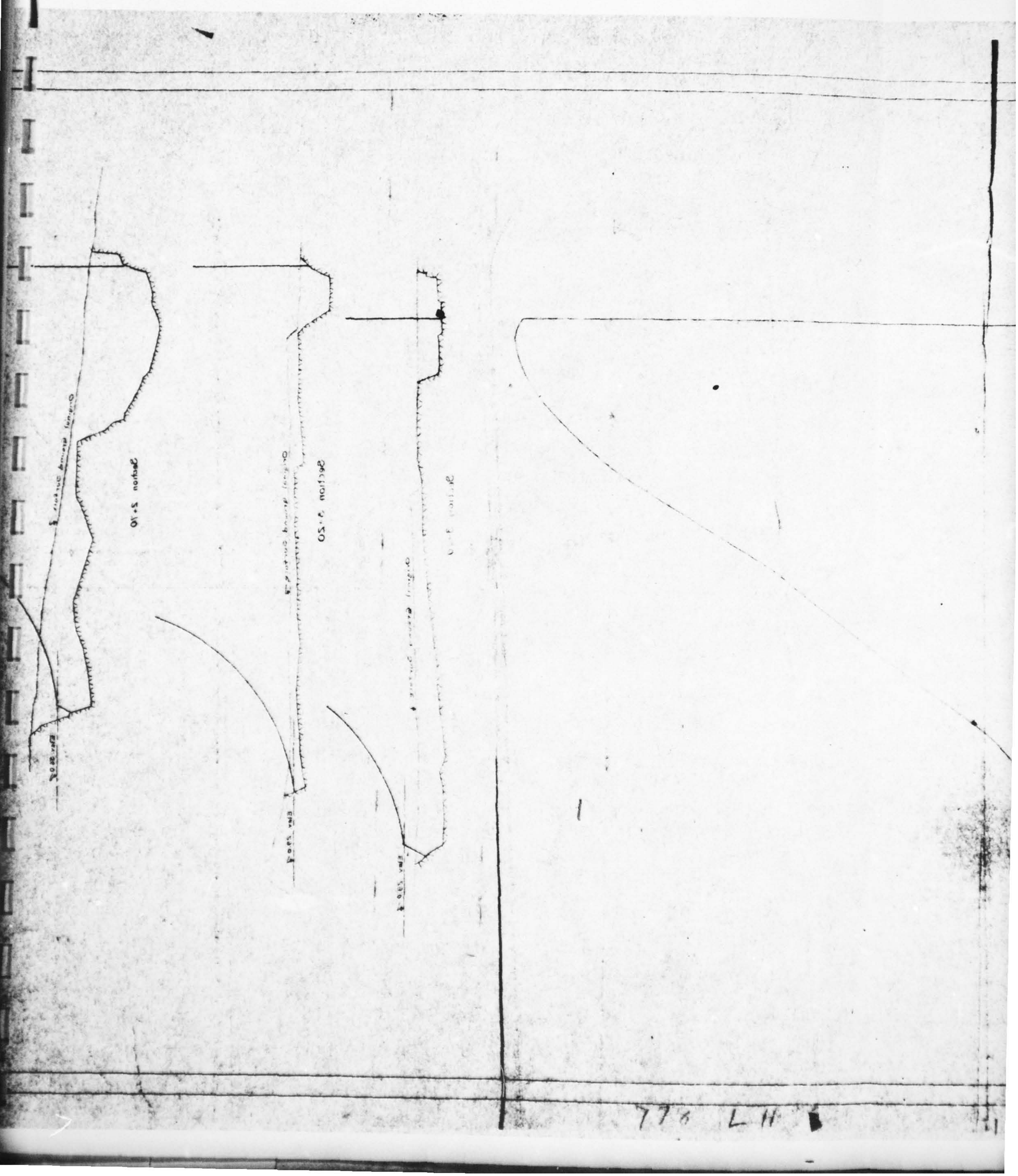
CONSTRUCTION DRAWINGS



H-5275-3
WALLKILL RIVER
Canal
Excavate to El. 26
Excavate to El. 98.0
Old Mill Bldg.
Diversion Walls
Approximate North
RECORD OF ISSUES AND REVISIONS
APPROVED



Tower #147
RIVER
Excavate to El. 81.5
UNITED HUDSON ELECTRIC CORPORATION
SITE-LOCATION OF STRUCTURES
THE J.G. WHITE ENGINEERING CORPORATION
EXCHANGE PLACE, N.Y.
SCALE
H-5875-3



Section 2+70
Section 3+20

AD-A069 101 KIMBALL (L ROBERT) AND ASSOCIATES EBENSBURG PA F/G 13/2
NATIONAL DAM SAFETY PROGRAM. STURGEON POOL DAM (INVENTORY NUMBE--ETC(U)
SEP 78 R J KIMBALL DACW51-78-C-0025

UNCLASSIFIED NL

3 OF 3

AD A069101

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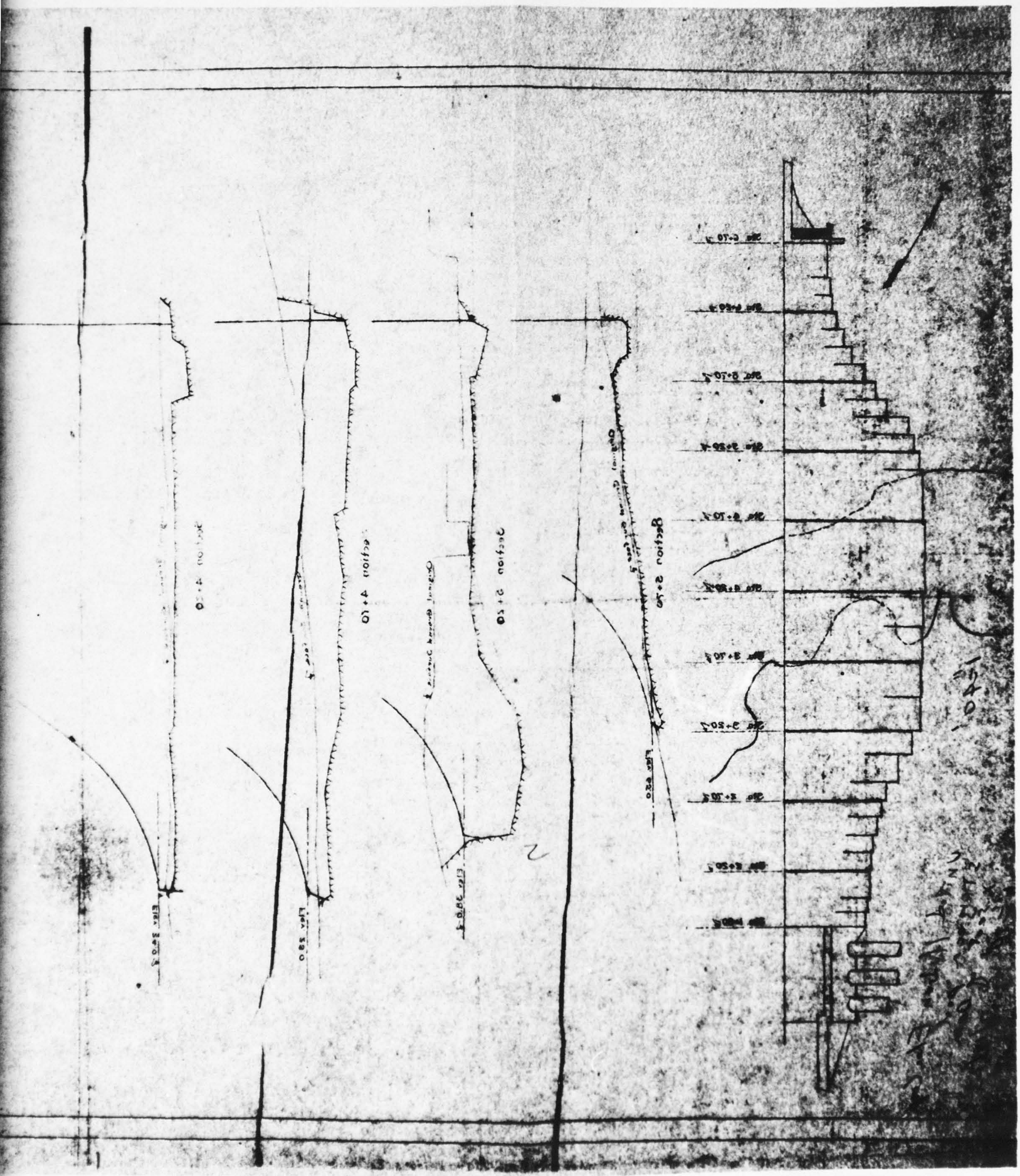
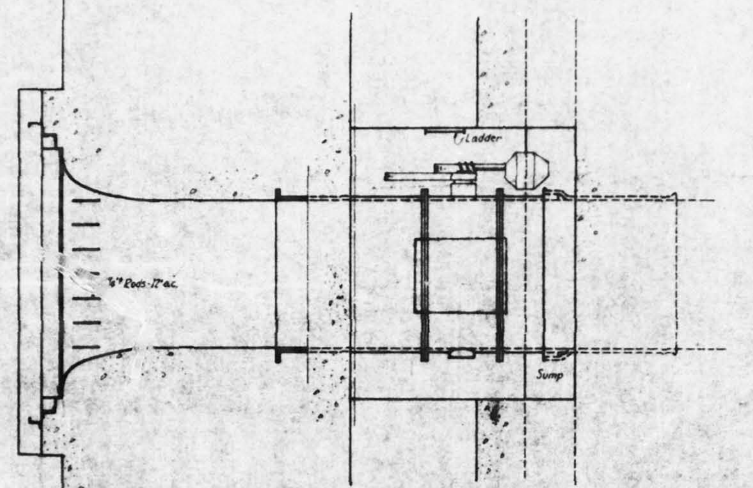
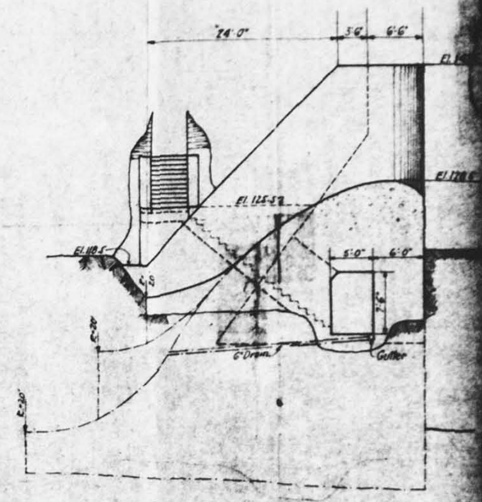
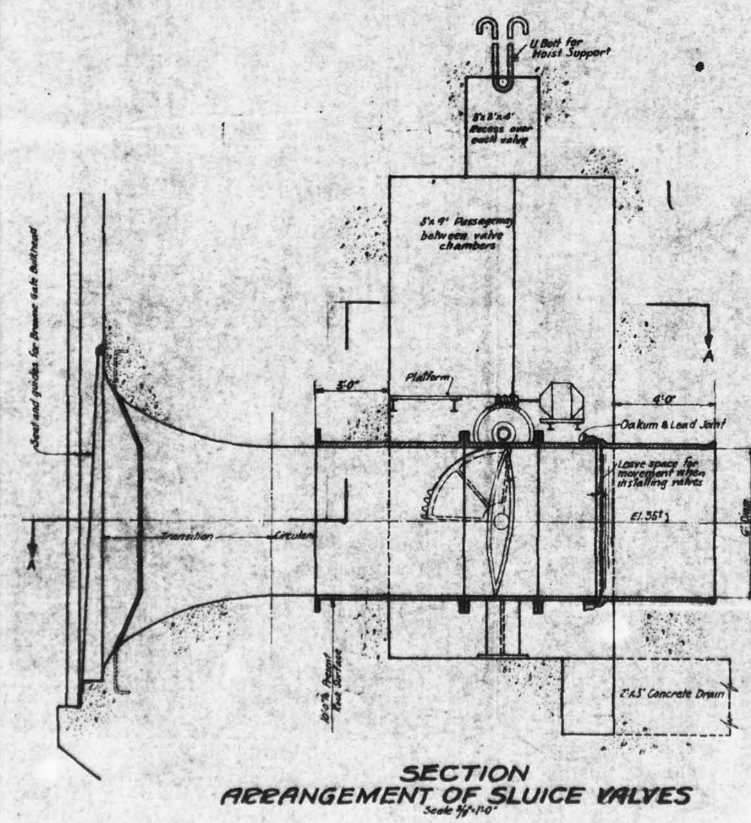




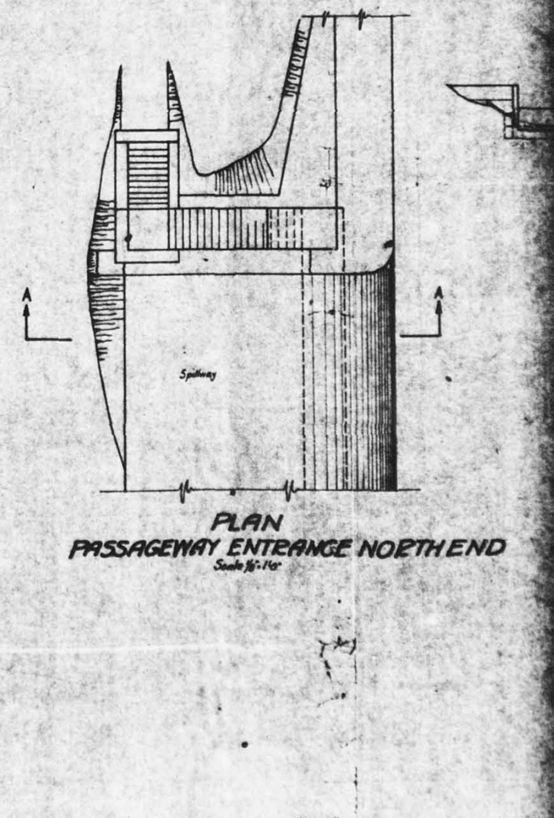
SECTION "A-A"



SECTION "A-A"

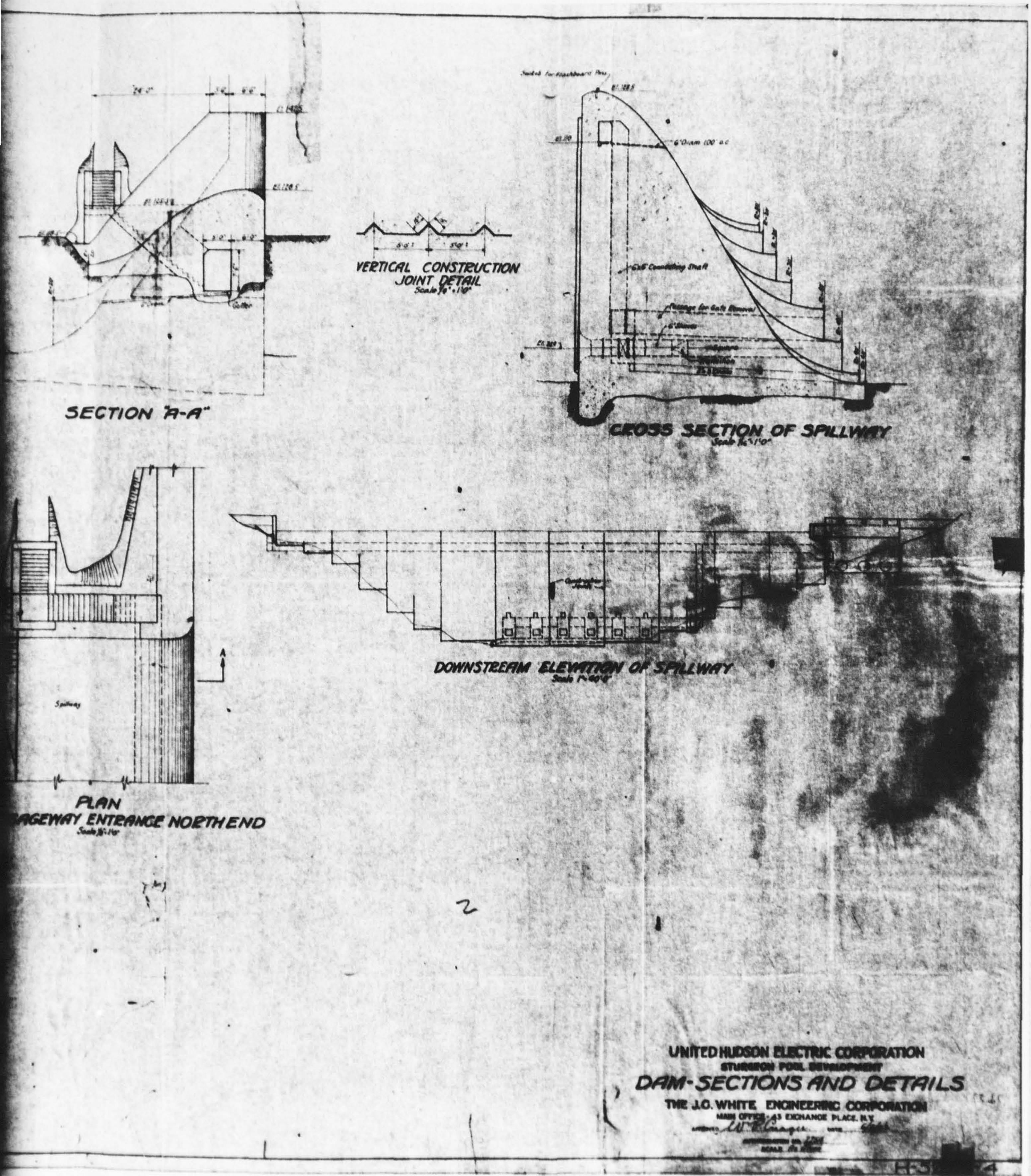


SECTION
ARRANGEMENT OF SLUICE VALVES
Scale ⅜"=1'-0"



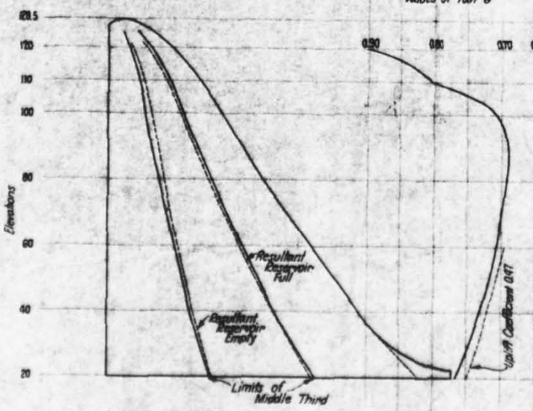
PLAN
PASSAGEWAY ENTRANCE NORTH END
Scale ⅛"=1'-0"

RECORD OF ISSUES AND REVISIONS

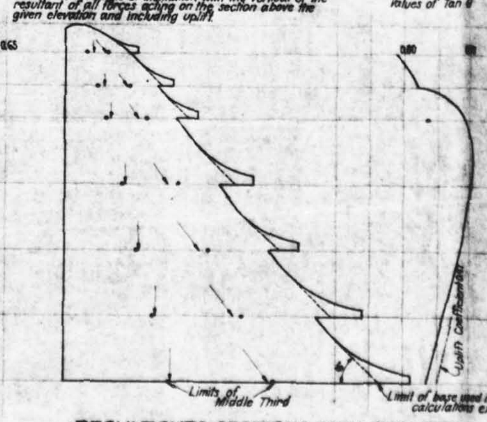

SECTION "A-A"
VERTICAL CONSTRUCTION JOINT DETAIL
CROSS SECTION OF SPILLWAY
Spillway
PLAN
AGEWAY ENTRANCE NORTH END
DOWNSTREAM ELEVATION OF SPILLWAY
UNITED HUDSON ELECTRIC CORPORATION
STURGEON POOL DEVELOPMENT
DAM-SECTIONS AND DETAILS
THE J.G. WHITE ENGINEERING CORPORATION
MAIN OFFICE - 43 EXCHANGE PLACE, N.Y.

H-5310-1

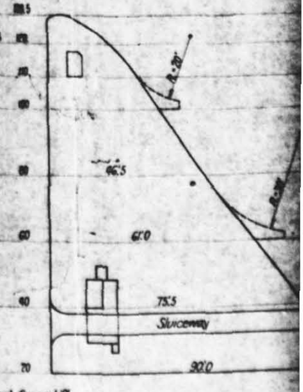
RECORD OF ISSUES AND REVISIONS



RESULTANTS MAXIMUM SECTION
Openings not shown

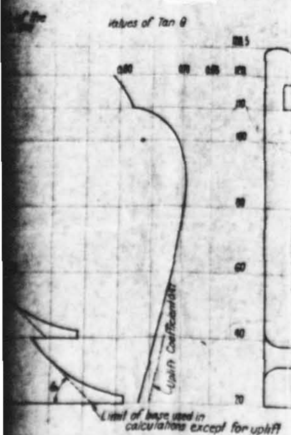


RESULTANTS SECTIONS WITH BUCKETS
Openings not shown

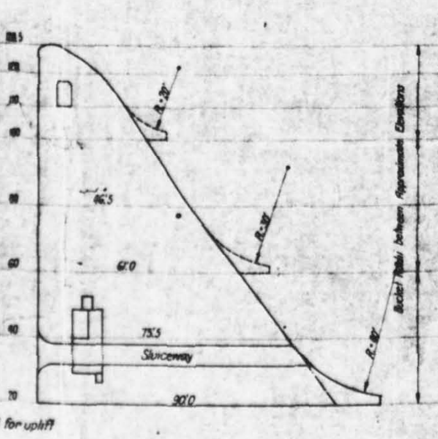


DIMENSION SEC
Note: Sluiceway
For dimensions of openings

SPILLWAY DAM



...S WITH BUCKETS

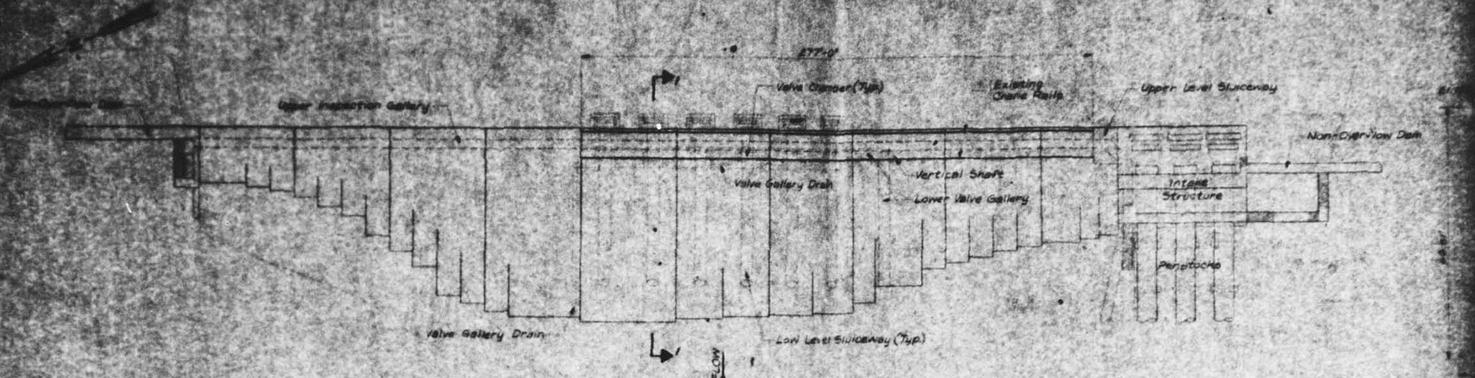


DIMENSION SECTION
Note Sluiceway 25' oc
For dimensions of openings, scale Drawing H-5276

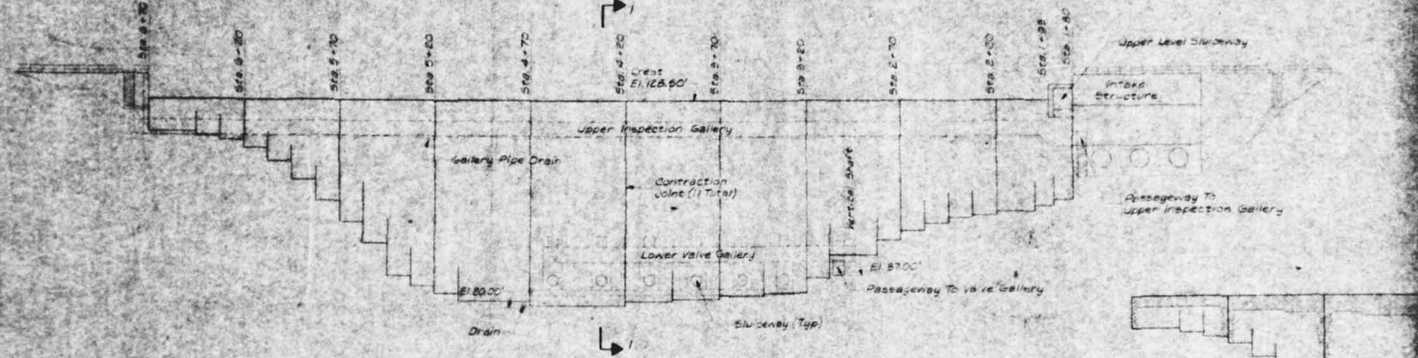
...Y DAM

UNITED H... ELECTRIC CORPOR...

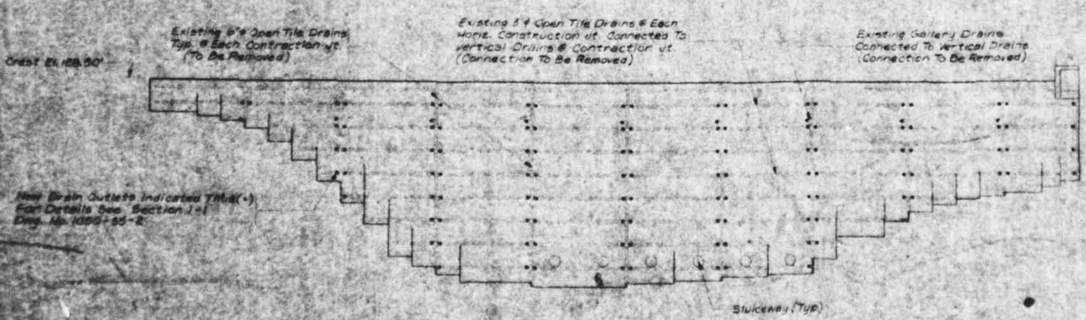
ANAL... ...ESSES IN...



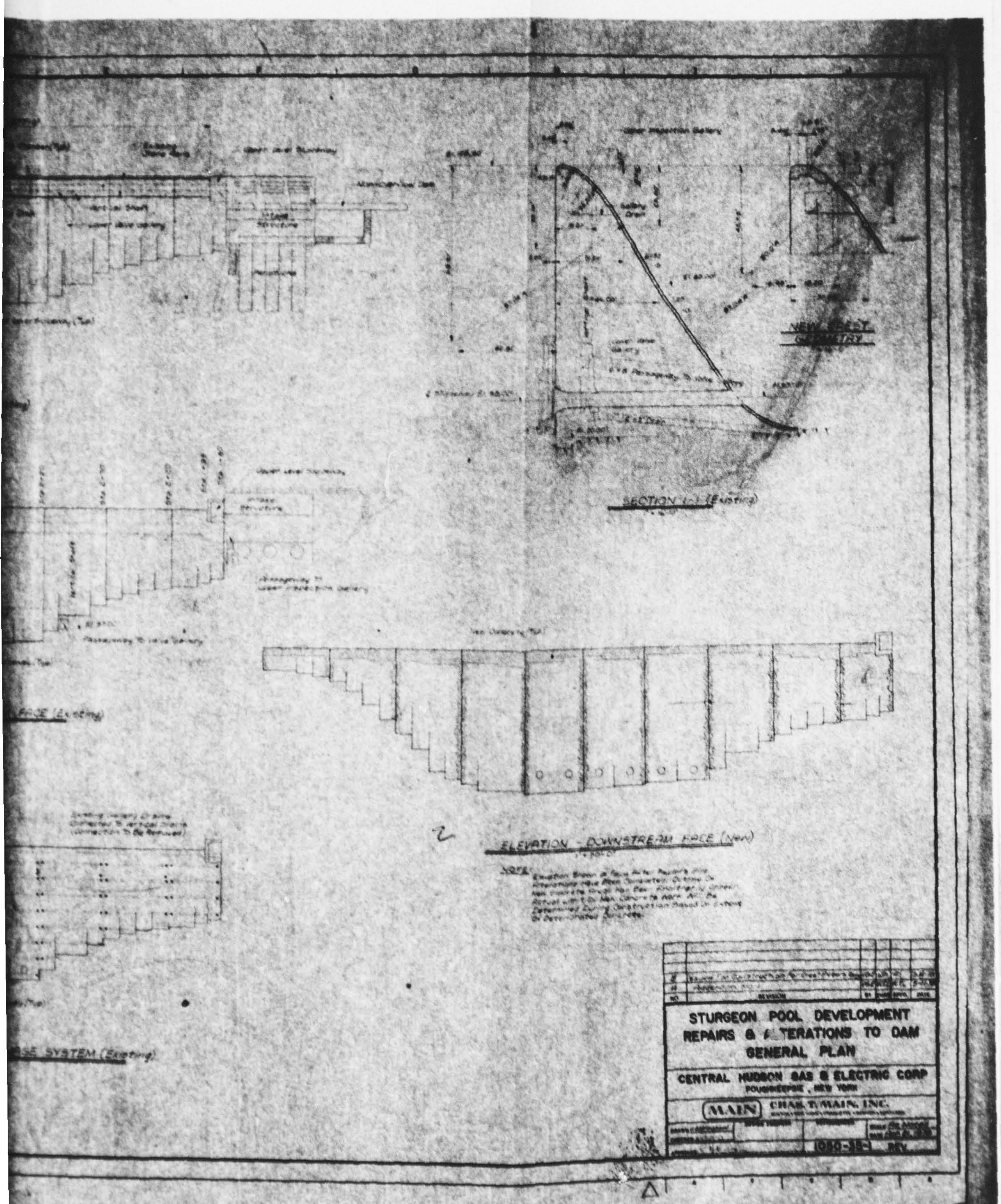
PLAN (Existing)
1" = 30'-0"

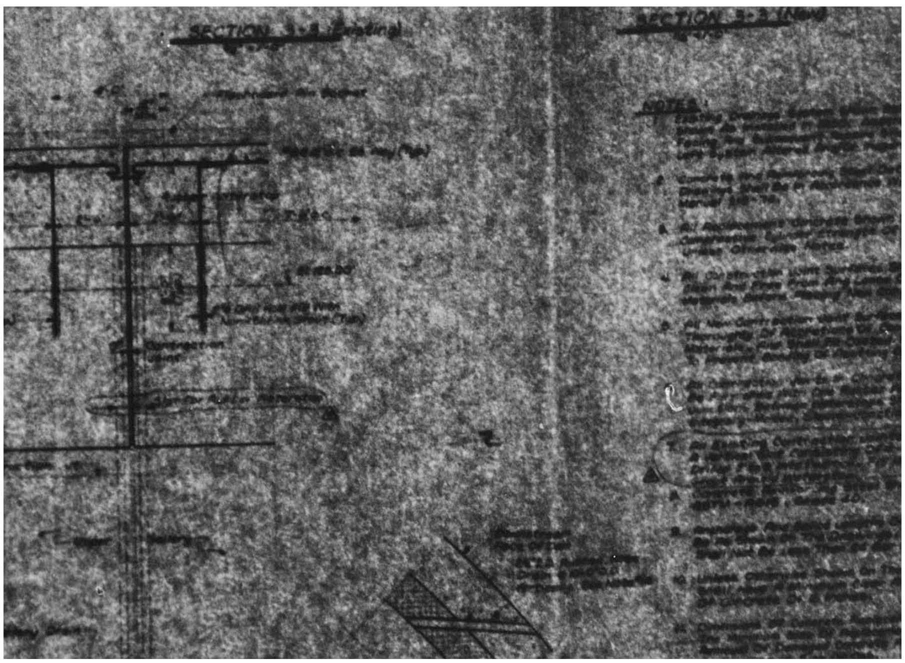


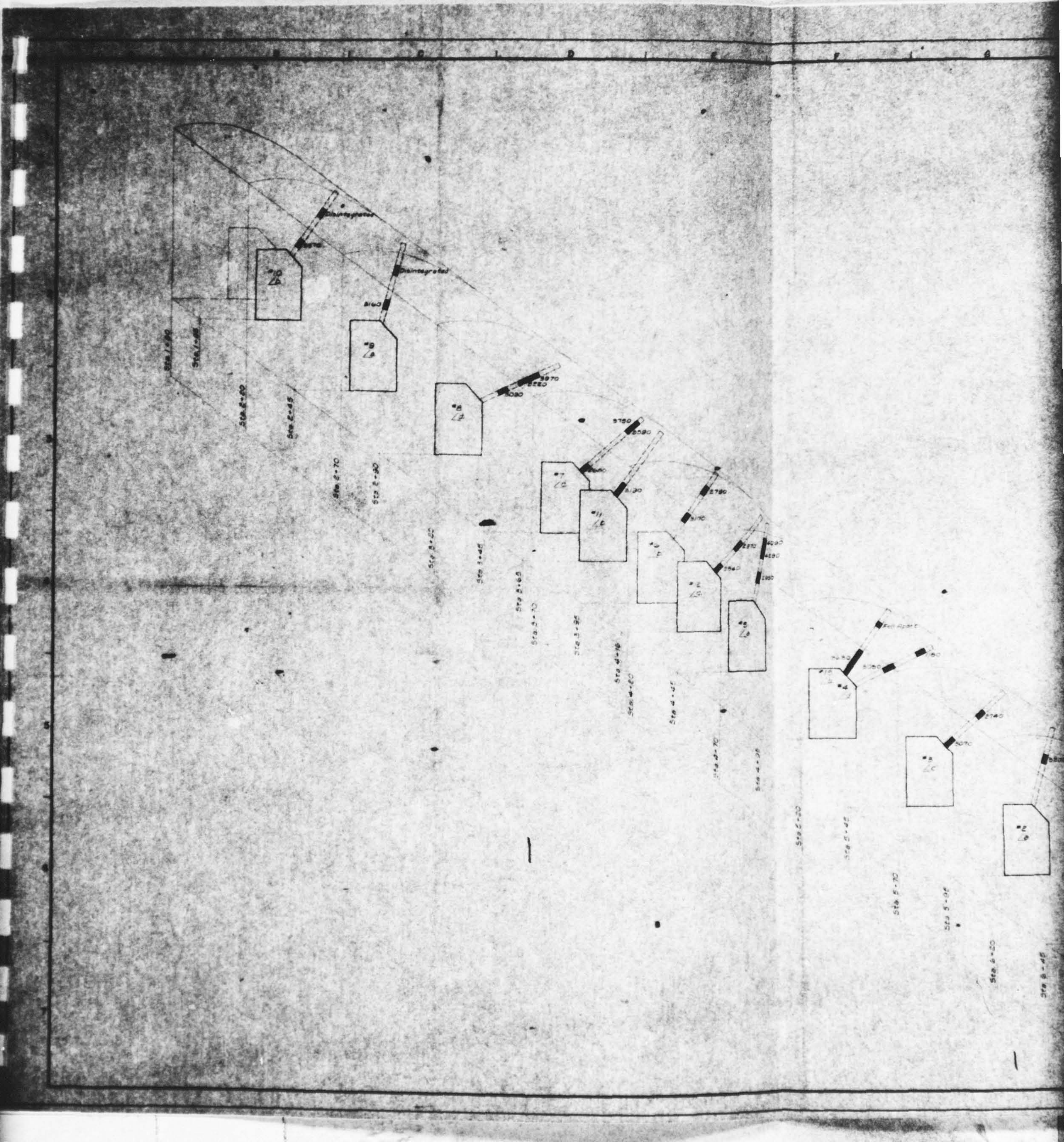
ELEVATION - DOWNSTREAM FACE (Existing)
1" = 30'-0"

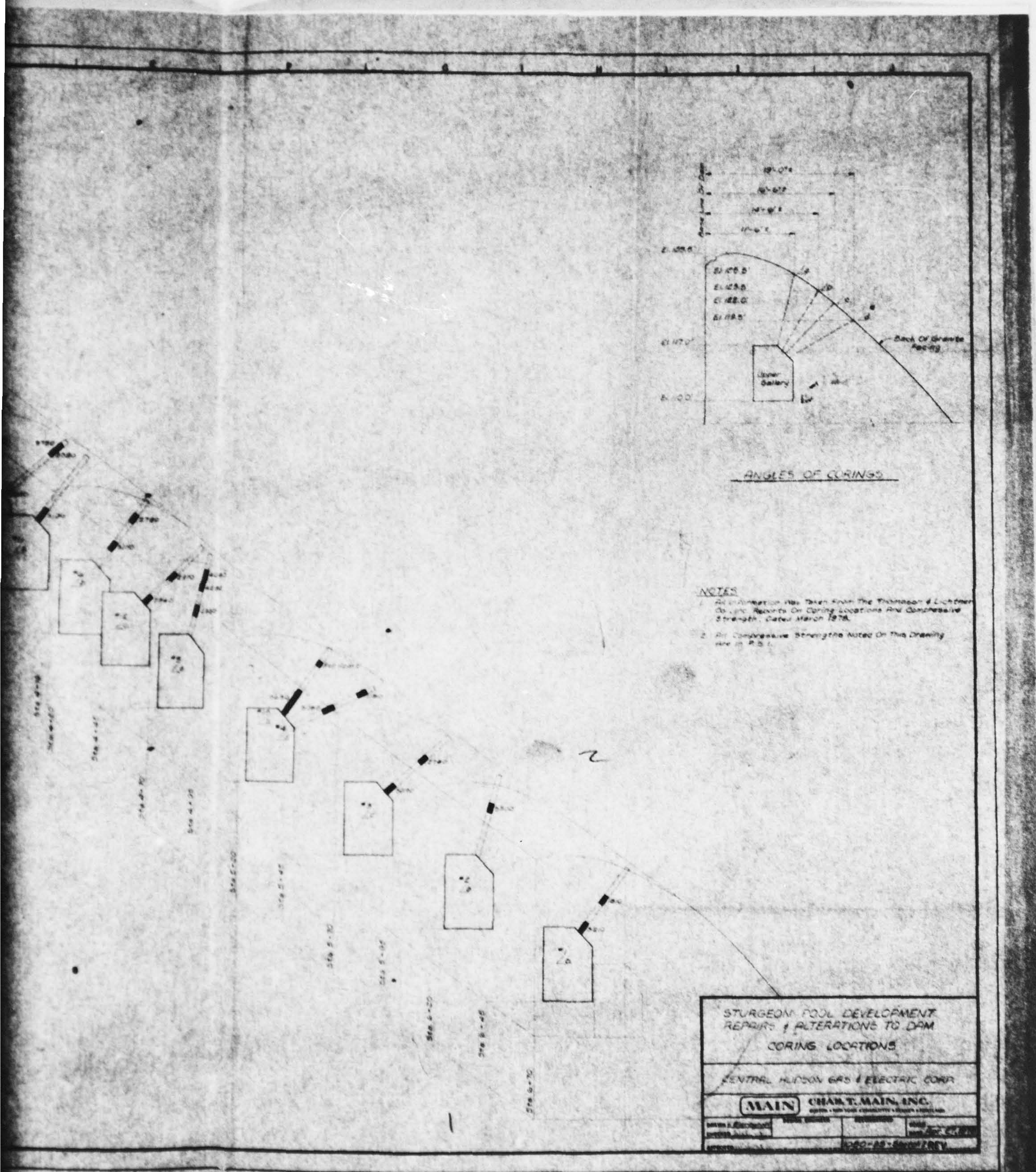


ELEVATION - DOWNSTREAM FACE DRAINAGE SYSTEM (Existing)
1" = 30'-0"


NEW CREST GEOMETRY
SECTION 1-1 (Existing)
FACE (Existing)
ELEVATION - DOWNSTREAM FACE (New)
NOTE:
SE SYSTEM (Existing)
REVISION
STURGEON POOL DEVELOPMENT
REPAIRS & ALTERATIONS TO DAM
GENERAL PLAN
CENTRAL HUDSON GAS & ELECTRIC CORP
POUGHKEEPSIE, NEW YORK
MAIN
CHAS. T. MAIN, INC.
REV


SECTION 3-3 (Existing)
SECTION 3-3 (New)
NOTES:


Disintegrated
Disintegrated



Upper Gallery
Back Of Granite Facing
ANGLES OF CORINGS
NOTES
STURGEON POOL DEVELOPMENT
REPAIRS & ALTERATIONS TO DAM
CORING LOCATIONS
CENTRAL HUDSON GAS & ELECTRIC CORP.
MAIN
CHAS. T. MAIN, INC.

APPENDIX F

VISUAL CHECK LIST

CHECK LIST
VISUAL INSPECTION
PHASE 1

NAME DAM Sturgeon Pool Dam COUNTY Ulster STATE New York ID# 75

TYPE OF DAM Concrete Gravity HAZARD CATEGORY High

DATE(s) INSPECTION August 29, 1978 WEATHER partly cloudy TEMPERATURE 80°

POOL ELEVATION AT TIME OF INSPECTION 108.0 M.S.L. TAILWATER AT TIME OF INSPECTION 20.0 M.S.L.

INSPECTION PERSONNEL:

R. Jeffry Kimball, P.E. - LRK	Peter Rimsa - Central Hudson	Tom Duffey - Central Hudson
James T. Hockensmith - LRK	Dick Dammier - Central Hudson	Phil Brown - C.T. Main

R. Jeffrey Kimball RECORDER

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	N/A	
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	N/A	
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	N/A	
VERTICAL AND HORIZONTAL ALINEMENT OF THE CREST	N/A	
RIPRAP FAILURES	N/A	

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	N/A	
ANY NOTICEABLE SEEPAGE	N/A	
STAFF GAGE AND RECORDER	N/A	
DRAINS	N/A	

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATION
ANY NOTICEABLE SEEPAGE	None noted	Water level lowered changing conditio from normal.
STRUCTURE TO ABUTMENT/EMBANKMENT JUNCTIONS	No problems noted at abutments - appeared to be well founded.	
DRAINS	Vertical and horizontal drains behind granite facing - Drains appear to be functioning improperly as some of the granite has been lifted due to freeze-thaw.	
WATER PASSAGES	Upper and lower galleries have drains	
FOUNDATION	Apparently sound rock	

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATION
SURFACE CRACKS CONCRETE SURFACES	Concrete surface covered with granite facing. The facing is deteriorating due to freeze thaw below the granite.	
STRUCTURAL CRACKING	None noted.	
VERTICAL AND HORIZONTAL ALIGNMENT	No deviations noted.	
MONOLITH JOINTS	Vertical construction joints - divided into ten separate sections - considerable seepage is reported through the joints - the upstream slope joints are reportedly open 8"to 10" in places.	
CONSTRUCTION JOINTS STAFF GAGE OF RECORDER:	Staff gage on dam - no continuous record.	

OUTLET WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	Outlet end of sluice tunnels observed no probelems noted.	
INTAKE STRUCTURE	The intake of the sluice gates is below water. The gates themselves are not operable. Additional outlet through the power penstacks in the left abutment section.	
OUTLET STRUCTURE	Sluice tunnels not observed penstocks reported to have seepage around base.	
OUTLET CHANNEL	Wallkill River to Rondout Creek - Rock bottom near the dam - Wooded downstream.	
EMERGENCY GATE	None on sluice gates Emergency gates on penstacks operated by crane on bulkhead - not operated in recent past.	

UNGATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR	Not applicable	See gated spillway for ogee outflow section.
APPROACH CHANNEL	N/A	
DISCHARGE CHANNEL	N/A	
BRIDGE AND PIERS	N/A	

GATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE SILL	Crest presently being removed and replaced with concrete	Ogee overflow section at center of dam
APPROACH CHANNEL	None	
DISCHARGE CHANNEL	Downstream face of dam on granite facing to Wallkill River flip buckets provided at abutment sections.	
BRIDGE AND PIERS	None	
GATES AND OPERATION EQUIPMENT	The ogee crest is gated with two level flashboards - Not in place during the inspection. The center third of the ogee section has 2.5' high flashboards The outer thirds have 3.5' high flash-boards. The boards are designed to fail with 1.5' of head over the low section.	

DOWNSTREAM CHANNEL

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	Rock bottom with vegetated slopes No major obstructions - confluence with Rondout Creek just downstream.	
SLOPES	Left slope gentle - right slope steep.	
APPROXIMATE NO. OF HOMES AND POPULATION	Power house at toe of dam. Approximately 30 dwellings along Roundout Creek within 2 to 3 miles.	

RESERVOIR

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	Gentle to steep - relatively stable.	
SEDIMENTATION	Reportedly a high sediment load carried to the reservoir by the Wallkill River.	

INSTRUMENTATION

VISUAL EXAMINATION	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS	None noted - some survey monuments must be present for current construction.	
OBSERVATION WELLS	None	
WEIRS	None	
PIEZOMETERS	None	
OTHER	Staff gage.	

APPENDIX G

ENGINEERING DATA CHECK LIST

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION
PHASE I

NAME OF DAM Sturgeon Pool

ID# 75

ITEM	REMARKS
AS-BUILT DRAWINGS	None - design drawings from owner and state.
REGIONAL VICINITY MAP	Acres American 1978 Report.
CONSTRUCTION HISTORY	No data available
TYPICAL SECTIONS OF DAM	Design Drawings - 1973 Main Report - 1978 Acres American Report.
OUTLETS - PLAN	Drawings Available
- DETAILS	Limited details available
- CONSTRAINTS	None
- DISCHANCE RATINGS	None
RAINFALL/RESERVOIR RECORDS	Not reviewed - apparently reservoir levels kept.

ITEM	REMARKS
DESIGN REPORTS	None available
GEOLOGY REPORTS	1978 Acres American Report did geologic study
DESIGN COMPUTATIONS	None available
HYDROLOGY & HYDRAULICS	None available
DAM STABILITY	1973 and 1978 Acres Reports conducted stability
SEEPAGE STUDIES	Discussions on seepage conditions and plans for repairs and controls
MATERIALS INVESTIGATIONS	None
BORING RECORDS	None
LABORATORY	Testing of concrete mortar cubes
FIELD	None
POST-CONSTRUCTION SURVEYS OF DAM	1973 C.T. Main Study 1978 Acres American Study 1978 C.T. Main Study
BORROW SOURCES	Not applicable

ITEM	REMARKS
MONITORING SYSTEMS	None
MODIFICATIONS	Presently modifying crest
HIGH POOL RECORDS	Reported max. W.E. 136' in 1955
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	1973 C.T. Main Study 1978 C.T. Main Study 1978 Acres American Study
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	None
MAINTENANCE OPERATION RECORDS	No formal records kept C.T. Main recommended a routine maintenance schedule

	REMARKS
SPILLWAY PLAN	Same as dam sections.
SECTIONS	
DETAILS	
OPERATING EQUIPMENT PLANS & DETAILS	Drawings available from owner

CHECK LIST
HYDROLOGIC AND HYDRAULIC
ENGINEERING DATA

DRAINAGE AREA CHARACTERISTICS: 787 square miles, wooded, cultivated

ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): 131.0 (approximately 8,000 ac-ft)

ELEVATION TOP FLOOD CONTROL POOL (STORAGE CAPACITY): N/A

ELEVATION MAXIMUM DESIGN POOL: apparently 141.3'

ELEVATION TOP DAM: 141.5'

CREST: Overflow Section

a. Elevation 128.5'
b. Type ogee overflow section
c. Width 4'
d. Length 490'
e. Location Spillover center of dam
f. Number and Type of Gates flashboards low level 131.0 - top 132.0

OUTLET WORKS:

a. Type Concrete tunnels with sluice gates - not operable
b. Location Center of dam
c. Entrance inverts 32'
d. Exit inverts 32'
e. Emergency draindown facilities Penstock now used

HYDROMETEOROLOGICAL GAGES:

a. Type None
b. Location None
c. Records None

MAXIMUM NON-DAMAGING DISCHARGE W.E. 136'